SETO

Environment Canada | Environnement Canada

Car | Intario Agreement on Great Lakes Water Quality

Ontario Ministry of the Environment

To Establish Viable Methods of Maintaining Waste Treatment Facility Efficiencies with Reference to Flow Variations

Volume II

Research Report No. 55

STANDARDS DEVELOPMENT BRANCH OMOE
3693600006538

rch Program for the Abatement of Municipal Pollution
nder Provisions of the Canada-Ontario Agreement
on Great Lakes Water Quality

TD
746
.E88
1977
MOE

Copyright Provisions and Restrictions on Copying:

This Ontario Ministry of the Environment work is protected by Crown copyright (unless otherwise indicated), which is held by the Queen's Printer for Ontario. It may be reproduced for non-commercial purposes if credit is given and Crown copyright is acknowledged.

It may not be reproduced, in all or in part, part, for any commercial purpose except under a licence from the Queen's Printer for Ontario.

For information on reproducing Government of Ontario works, please contact Service Ontario Publications at copyright@ontario.ca

CANADA-ONTARIO AGREEMENT

RESEARCH REPORTS

These RESEARCH REPORTS describe the results of investigations funded under the Research Program for the Abatement of Municipal Pollution within the provisions of the Canada-Ontario Agreement on Great Lakes Water Quality. They provide a central source of information on the studies carried out in this program through in-house projects by both Fisheries and Environment Canada, and the Ontario Ministry of Environment, and contracts with municipalities, research institutions and industrial organizations.

The Scientific Liaison Officer for this project was Mr. B.I. Boyko, Ontario Ministry of the Environment.

Enquiries pertaining to the Canada-Ontario Agreement RESEARCH PROGRAM should be directed to -

Wastewater Technology Centre
Canada Centre for Inland Waters
Fisheries and Environment Canada
P.O. Box 5050
Burlington, Ontario L7R 4A6

Ontario Ministry of the Environment
Pollution Control Branch
135 St. Clair Avenue West
Toronto, Ontario M4V 1P5

TD
746
.E88
1977

To establish viable methods of maintaining waste treatment facility efficiencies with reference to flow variations :

78482

TO ESTABLISH VIABLE METHODS OF MAINTAINING WASTE TREATMENT FACILITY EFFICIENCIES WITH REFERENCE TO FLOW VARIATIONS

VOLUME II

by

James F. MacLaren Limited
Environmental Engineers and Scientists

RESEARCH PROGRAM FOR THE ABATEMENT
OF MUNICIPAL POLLUTION WITHIN THE
PROVISIONS OF THE CANADA-ONTARIO
AGREEMENT ON GREAT LAKES WATER QUALITY

Project No. 72-5-10

This document may be obtained from -

Training and Technology Transfer
Division (Water)
Environmental Protection Service
Fisheries and Environment Canada
Ottawa, Ontario
K1A 0C8

Ontario Ministry of the Environment
Pollution Control Branch
135 St. Clair Avenue West
Toronto, Ontario
M4V 1P5

REVIEW NOTICE

This report has been reviewed by the Technical Committee of the Canada-Ontario Agreement on Great Lakes Water Quality and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of either the Ontario Ministry of the Environment or Fisheries and Environment Canada, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

Minister of Supply and Services Canada 1977
Cat. No. En43-11/55

ISBN 0-662-00728-X

ABSTRACT

This volume is a continuation of the study reported in Research Report No. 11, published in 1974. At that time, it was pointed out that equalization of sewage flow variations would have several potential benefits with respect to maintaining efficiencies in waste treatment facilities. In the earlier report alternatives to bypassing were studied, and a comprehensive methodology was developed to assess flow variations and predict the effect of cyclic diurnal flow on the performance of each waste treatment process.

In this study sewage flow records from eleven individual sewage systems of varying sizes were examined. Daily flow records have been evaluated for all plants to establish variations in average daily flow over periods of one, two, and in some cases three years. Flow records were statistically analyzed to determine the flow rate occurring at the 10 and 25 percent exceedence levels. Typical exceedence flow profiles for two municipal waste treatment plants are presented, along with the mean daily flow, the maximum flow and the ten and 25 percent exceedence levels.

To establish meaningful design criteria for equalization, extensive analysis of daily and diurnal variations of flow in Ontario sewage treatment plants was carried out. A significant conclusion of the analysis was that the peaking factors are essentially independent of the plant size.

The most important criteria for flow equalization appeared to be the maximum daily peaking factor and the diurnal peaking factor. Diurnal peaking factors decreased with increasing peaking periods.

On the basis of the findings a new and simplified method is suggested for sizing and operation of equalization basins. Maximum benefit due to equalization can be predicted for the performance of primary clarifiers.

A construction cost comparison for plant sizes of 5 and 50 mgd indicates that fully equalized flow can result in a small reduction over variable flow design. Partial equalization does not appear to be a viable alternative. The cost benefits may be considered to be conservative, yet potential cost benefits may be higher when increased performance of the various units and the elimination of surcharge energy costs at peak loadings are included.

RÉSUMÉ

Le présent document fait suite à l'étude publiée en 1974 (Rapport de recherche n° 11) où nous indiquions que la régularisation du débit des eaux usées pourrait notablement accroître l'efficacité des usines d'épuration. Nous avions alors étudié les options de rechange à la dérivation et élaboré une méthode globale pour évaluer les variations de débit et prévoir l'effet d'un débit diurne cyclique sur le rendement de chaque procédé de traitement.

Nous examinons maintenant le débit mesuré dans 11 installations d'épuration de capacité diverse. L'estimation du débit journalier de chacune permet d'établir les variations moyennes quotidiennes sur une période d'un, deux et, dans certains cas, trois ans. L'analyse statistique des débits a permis de déterminer leurs niveaux excédentaires de 10 et de 25 p. 100. Nous montrons le profil des débits excédentaires de deux usines municipales ainsi que leur rendement moyen journalier, leur débit maximal et leurs débits excédentaires de 10 et de 25 p. 100.

On a analysé à fond les variations diurnes et journalières du débit des usines d'épuration de l'Ontario pour établir des critères de conception propres à améliorer la régularisation; une importante conclusion s'est dégagée de cette analyse: les coefficients de pointe sont tout à fait indépendants de la capacité de l'usine.

Les critères les plus utiles à la régularisation semblent être les coefficients de pointe maximal journalier et diurne. Ce dernier diminue à mesure qu'augmente la période de pointe.

Les résultats obtenus permettent de proposer une nouvelle méthode simplifée de fixer les dimensions des bassins de régularisation et de les exploiter. On peut prévoir que la régularisation permettra de tirer le maximum d'avantages des clarificateurs primaires.

Si l'on compare les coûts de construction d'usines de 5 et de 50 millions de gallons par jour, on constate que la régularisation totale peut réduire quelque peu les coûts par rapport à une situation de débit variable. La régularisation partielle ne semble pas rentable. On peut considérer modérée l'estimation des avantages financiers, mais ils peuvent être supérieurs si l'on augmente le rendement des diverses étapes et si l'on élimine la demande excessive d'énergie qu'entraînent les charges de pointe.

Les résultats obtenus permettent de proposer une nouvelle méthode simplifée de fixer les dimensions des bassins de régularisation et de les exploiter. On peut prévoir que la régularisation permettra de tirer le maximum d'avantages des clarificateurs primaires.

Si l'on compare les coûts de construction d'usines de 5 et de 50 millions de gallons par jour, on constate que la régularisation totale peut réduire quelque peu les coûts par rapport à une situation de débit variable. La régularisation partielle ne semble pas rentable. On peut considérer modérée l'estimation des avantages financiers, mais ils peuvent être supérieurs si l'on augmente le rendement des diverses étapes et si l'on élimine la demande excessive d'énergie qu'entraînent les charges de pointe.

TABLE OF CONTENTS

		Page
ABSTRACT		i
TABLE OF CONTENTS		iv
	List of Figures	v
	List of Tables	vii
1.	INTRODUCTION	1
2.	LITERATURE REVIEW	2
3.	VARIATION IN RAW SEWAGE FLOW	4
3.1	Daily Flow Variations	4
3.2	Diurnal Flow Variations	7
3.2.1	The change in diurnal peaking factors (PFd) with dry and wet weather flows	11
3.2.2	Variation in diurnal peaking factors (PFd) relative to sewage flow	11
3.3	Combination of Daily and Diurnal Peaking Factors	15
4.	COMPARISON OF DESIGN CRITERIA AND COST OF WPCD	36
4.1	Design Criteria	36
4.2	Sizing of Equalization Basins	36
4.3	Primary Sedimentation	41
4.4	Aeration Tanks	42
4.5	Air Blowers (Aeration)	45
4.6	Return Sludge Pumping	45
4.7	Secondary Sedimentation Tanks	46
4.8	Chlorination	47
4.9	Cost Comparisons	48
4.10	Relative Operating Cost	48
5.	CONCLUSIONS	52
REFERENCES		54
APPENDIX I	Nomenclature	57
Appendix II	Analysis of Daily Flow Variations	61

LIST OF FIGURES

Figure		Page
1	The Dependence of the Maximum Flow on the Daily Mean Flow	8
2	The Dependence of the 10% Exceedence Level on the Mean Daily Flow	9
3	The Dependence of the 25% Exceedence Level on the Mean Daily Flow	10
4	The Dependence of the Peak Flow on Maximum Average Daily Flow	16
5	The Dependence of the 1-Hour Peak Flow on the Maximum Average Daily Flow	17
6	The Dependence of the 4-Hour Peak Flow on the Maximum Average Daily Flow	18
7	The Dependence of the 6-Hour Peak Flow on the Maximum Average Daily Flow	19
8	The Dependence of the 12-Hour Peak Flow on the Maximum Average Daily FLow	20
9	The Dependence of the Peak Flow on the 10% Exceedence Daily Flow	21
10	The Dependence of the 1-Hour Peak Flow on the 10% Exceedence Daily Flow	22
11	The Dependence of the 4-Hour Peak Flow on the 10% Exceedence Daily Flow	23
12	The Dependence of the 6-Hour Peak Flow on the 10% Exceedence Daily Flow	24
13	The Dependence of the 12-Hour Peak Flow on the 10% Exceedence Daily Flow	25
14	The Dependence of the Maximum Flow on the 25% Exceedence Daily Flow	26
15	The Dependence of the 1-Hour Peak Flow on the 25% Exceedence Daily Flow	27
16	The Dependence of the 4-Hour Peak Flow on the 25% Exceedence Daily Flow	28

LIST OF FIGURES (CONT'D)

Figure		Page
17	The Dependence of the 6-Hour Peak Flow on the 25% Exceedence Daily Flow	29
18	The Dependence of the 12-Hour Peak Flow on the 25% Exceedence Daily Flow	30
19	The Dependence of the Maximum Flow on the Average Daily Flow	31
20	The Dependence of the 1-Hour Peak Flow on the Average Daily Flow	32
21	The Dependence of the 4-Hour Peak Flow on the Average Daily Flow	33
22	The Dependence of the 6-Hour Peak Flow on the Average Daily Flow	34
23	The Dependence of the 12-Hour Peak Flow on the Average Daily Flow	35
24	Idealized Instantaneous FLow Variation	38
25	Frequency of Exceedence of Daily Mean Flows Municipality: D Year: 1971	62
26	Frequency of Exceedence of Daily Mean Flows Municipality: D Year: 1972	63
27	Frequency of Exceedence of Daily Mean Flows Municipality: D Year: 1973	64
28	Frequency of Exceedence of Daily Mean Flows Municipality: E Year: 1970	65
29	Frequency of Exceedence of Daily Mean Flows Municipality: E Year: 1971	66
30	Frequency of Exceedence of Daily Mean Flows Municipality: E Year: 1972	67

LIST OF TABLES

Table		Page
1	Daily Flow Characteristics of Treatment Plants	5
2	Daily Peaking Factors (PF)	6
3	Average Parameter Values and 95 Percent Confidence Levels - Daily Flow Variations	6
4	Diurnal Peaking Factors (PFd) during Maximum Flow Conditions	12
5	Diurnal Peaking Factors During 10% Exceedence Flow Conditions	12
6	Diurnal Peaking Factors During 25% Exceedence Flow Conditions	13
7	Diurnal Peaking Factors during Mean Daily Flow Conditions	13
8	Average Diurnal Peaking Factors for Various Peaking Periods	14
9	The Dependence of the Characteristic Constants of Diurnal Peak Flows on the Peaking Period	14
10	Equalization Basin Volumes for 5 and 50 mgd Sewage Treatment Plants	40
11	Design Criteria for Primary Sedimentation Tanks	43
12	Aeration Tank Volumes for 5 mgd and 50 mgd Plants	44
13	Aeration Requirement for 5 mgd and 50 mgd Plants	45
14	Return Sludge Pumping Capacity	46
15	Secondary Sedimentation Tank Area	47
16	Chlorination Basin Size	47
17	Estimated Construction Cost 5 mgd Plant	49
18	Estimated Construction Cost 50 mgd Plant	50

1. INTRODUCTION

Sewage flows in municipal sewage systems typically exhibit diurnal, daily and seasonal variations. The flow variability is in part attributable to non-uniformity in domestic water usage, industrial discharges, storm runoff and the physical condition of the sewer lines. In the past sewage treatment facilities have been commonly designed on the basis of treating the average daily flow with provision for handling a portion of the normal flow fluctuations. During periods of extreme high flow, however, flows exceeding a certain level would be allowed to bypass one or more treatment stages, resulting in the discharge of partially treated effluent. Unregulated sewage bypass negates current efforts to upgrade the dry weather treatment of sewage.

In a previous report by James F. MacLaren Limited [1], alternatives to bypassing were studied. A comprehensive methodology was developed to assess flow variations and predict the effect of cyclic diurnal flow on the performance of each waste treatment process. The approach used in the study allowed economical analysis of capital cost. The two proposed options were: the design of a sewage treatment plant for near maximum flow conditions, and the incorporation of flow equalization in either existing or future waste treatment facilities.

2. LITERATURE REVIEW

Published data with reference to the effect of equalization basins on the design and performance of conventional secondary sewage treatment plants are few [1, 2, 3, 5]. In addition, a methodology for equalization basin design and operation has received very little attention in the past.

Methodologies for forecasting hydraulic profiles in raw sewage flow have been suggested by LaGrega and Keenan [2], and Smith [3]. LaGrega and Keenan used the Box-Jenkins technique to forecast diurnal flow profiles. This information was then used for the manipulation of flow rates to and from the equalization basin to ensure near steady state operation. Smith et al [3] assumed a sinusoidal diurnal flow pattern, and developed an empirical relationship between peak and average diurnal flows under average flow conditions. The model was applied to determine the maximum equalization volume needed on the basis of average daily flows. The volume was obtained by numerical integration of the presumed mathematical model, and it was found to be 12 percent of mean annual daily flow ($\bar{Q}_{MADF}$). However, a safety factor of 1.25 was used to raise this value to 15 percent. The equalization of flow prior to treatment would result in significant cost saving, more constant loading conditions, better BOD (biochemical oxygen demand) removal efficiency and improved solids settleability.

The magnitude of flow variation that may be allowed before the benefits of equalization are lost has not been established. However, LaGrega and Keenan [2] suggested an arbitrary maximum step change of 20 percent of the average daily flow.

Variation in sewage quality between the inflow and outflow to an equalization basin in terms of SS (suspended solids) and BOD was measured by LaGrega and Keenan [2]. Smith et al [3] calculated the quality variation assuming that the equalization basin was equivalent to a completely mixed biological reactor. LaGrega and Keenan established that, due to inadequate mixing, approximately 20 percent removal of SS occurred in the equalization basin. On the other hand, the BOD was found to increase. This was attributed to partial solubilization of suspended solids and consequent microbial population increase, since the soluble organic material did not increase significantly.

The quality variations in raw sewage were analyzed both in terms of diurnal and daily fluctuations as represented by BOD and SS. The pollutant loading estimated from discrete samples taken at various points on the diurnal curve, when weighted according to flow, yielded data quite different from that determined for composite daily samples. The diurnal patterns in SS and BOD concentrations paralleled the general diurnal flow pattern.

The capital cost benefit of flow equalization as compared to conventional design permitting bypassing was found to be negligible [3]. In comparison to a plant designed for maximum flow without bypassing, however, flow equalization was shown to be less expensive in both construction and operating costs. The findings were qualified by pointing out that the cost saving might be due to the assumptions used.

Smith et al [3] compared primary treatment performance under variable and equalized flow conditions. The SS removal was distinctly higher (47 percent as compared to 23 percent).

The location of an equalization basin within a treatment system was discussed by LaGrega and Keenan [2]. They suggested several alternatives including installation preceding the primary and the secondary treatment facility, but did not note any relative advantages.

The previous studies also established the need for comprehensive data analysis concerning flow and waste load fluctuations in typical sanitary sewage systems. These are particularly important to establish design criteria for waste flow equalization practices. This report serves as an extension of the previous study to further analyze waste flow fluctuations and establish a basis for sizing equalization basins for design purposes.

3. VARIATION IN RAW SEWAGE FLOW

3.1 Daily Flow Variations

Sewage flow records from eleven individual sewage systems of varying sizes were examined. The plants were selected in conjunction with the Ontario Ministry of the Environment. The goal was to establish the relationships between daily flow variations and the annual mean daily flow. The size of the system varied from those having mean daily sewage flows of approximately one million gallons per day (mgd) to 50 mgd. The degree of segregation of the sewer system (i.e. separate and combined sewers) also varied considerably from completely combined to essentially separate sewers.

Daily flow records were evaluated for all plants to establish variations in average daily flow over periods of one year. In most cases, complete flow records for at least a two-year period, and in several instances a three-year period, were included in the evaluation.

The flow records for each plant were statistically analyzed to determine the flow rate occurring at the 10 and 25 percent exceedence levels (i.e. the flow that is exceeded 10 and 25 percent of the time, respectively). Typical exceedence flow profiles for two municipal waste treatment plants are shown in Appendix II. The mean daily flow, together with the maximum flow and the 10 percent and 25 percent exceedence levels, are presented in Table 1.

The daily Peaking Factors (PF) were calculated from the ratios of the respective exceedence levels to the mean daily flow. The results are summarized in Table 2. They represent the range that may be found in actual sanitary sewer systems.

Although a correlation seems to exist between exceedence flows and daily mean flows, the individual peaking factors can vary considerably. The variability reflects on the particular characteristics of the individual sewer systems. For example, municipalities G, J and K are known to have combined sewer systems. It is apparent that in the absence of in-system bypassing, combined sewer systems yield consistently higher peaking factors due to storm flow; on the other hand, persistently high infiltration rates appear to result in flow equalization as observed in municipality C. It is evident from the wide variability in Maximum Day Peaking Factors

TABLE 1. DAILY FLOW CHARACTERISTICS OF TREATMENT PLANTS

Municipality		Maximum* Daily Flow (mgd)	10% Exceedence Flow (mgd)*	25% Exceedence Flow (mgd)*	Mean Daily Flow (mgd)*	Maximum Peak Ratio
A	1971	10.1	9.0	8.4	7.7	1.31
	1972	20.0	12.2	11.5	10.7	1.87
B	1971	10.6	7.01	6.1	5.5	1.93
	1972	13.4	10.6	9.4	7.6	1.76
	1973	15.2	12.2	10.8	9.7	1.57
C	1/6/71 to 31/5/72	1.9	1.7	1.6	1.5	1.27
	1/6/72 to 31/5/73	1.9	1.7	1.6	1.5	1.27
D	1971	14.2	9.39	8.4	7.8	1.82
	1972	13.2	8.63	8.0	7.3	1.81
	1973	12.7	8.82	7.9	7.1	1.79
E	1970	71.4	57.3	54.7	51.3	1.39
	1971	88.0	66.5	62.3	58.4	1.51
	1972	99.8	77.6	73.2	68.7	1.45
F	1970	19.5	13.5	12.7	11.9	1.64
	1971	20.1	16.3	15.2	14.1	1.43
	1972	29.4	19.5	18.4	17.6	1.67
G	1971	2.4	1.7	1.3	1.1	2.18
	1972	2.5	2.1	1.7	1.4	1.79
H	1968	93.4	66.2	48.7	40.1	2.33
	1969	100.0	77.0	58.8	44.4	2.25
	1970	100.0	72.2	56.2	45.0	2.22
I	1971	9.2	6.5	5.8	5.2	1.79
	1972	8.2	6.7	6.1	5.6	1.46
J	1972	32.5	30.8	25.4	20.5	1.59
K	1972	11.3	6.4	4.5	3.6	3.14

*(X 3785 cu m/day)

TABLE 2. DAILY PEAKING FACTORS (PF)

Municipality	Year	Mean Annual Daily Flow	Max. Day	10% Level	25% Level
A	1971	7.7	1.312	1.169	1.091
	1972	10.7	1.869	1.140	1.075
B	1971	5.5	1.922	1.284	1.113
	1972	7.6	1.752	1.399	1.238
	1973	9.7	1.570	1.259	1.118
C	'71-'72	1.5	1.234	1.110	1.045
	'72-'73	1.5	1.220	1.079	1.037
D	1971	7.8	1.835	1.208	1.085
	1972	7.3	1.819	1.187	1.098
	1973	7.1	1.791	1.244	1.107
E	1970	51.3	1.840	1.156	1.066
	1971	58.4	1.800	1.140	1.067
	1972	68.7	1.970	1.128	1.065
F	1970	11.9	1.632	1.122	1.064
	1971	14.1	1.426	1.156	1.075
	1972	17.6	1.672	1.109	1.047
G	1971	1.1	2.143	1.509	1.161
	1972	1.4	1.759	1.475	1.191
H	1968	40.1	2.330	1.654	1.219
	1969	44.4	2.248	1.737	1.320
	1970	45.0	2.222	1.603	1.246
I	1971	5.2	1.768	1.246	1.107
	1972	5.6	1.476	1.207	1.092
J	1972	20.5	3.537	1.502	1.241
	1973	27.0	2.743	1.574	1.239
K	1972	3.6	3.165	1.796	1.258
	1973	5.2	3.996	1.512	1.199

TABLE 3. AVERAGE PARAMETER VALUES AND 95 PERCENT CONFIDENCE LEVELS - DAILY FLOW VARIATIONS

Parameter	Maximum Flow Average	Maximum Flow Interval	10% Exceedence Average	10% Exceedence Interval	25% Exceedence Average	25% Exceedence Interval
a	1.7	3.1 1.2	1.3	1.7 1.14	1.12	1.27 1.05
n	1.02	1.06 0.98	1.00	1.03 0.97	1.00	1.01 0.99

that caution must be exercised in the selection of a peaking factor for a particular municipality.

The maximum 10 percent and 25 percent exceedence level flows are plotted against the mean daily flow on Figures 1, 2 and 3. The general equation for the best fit at data points is:

$$y = a\ x^{n}$$

where: y = daily flow for each respective exceedence level

a, n = characteristic parameters

x = annual mean daily flow.

From the Figures it may be concluded that the data conforms to the above equation.

The parameter values were calculated by regression analysis. These, together with the 95 percent confidence intervals for each exceedence level, are presented in Table 3. Generally, the value of "n" is very near unity. In this case the parameter "a" corresponds to the mean Peaking Factor for a particular exceedence level. The implication is that the PF is independent of the mean daily flow. In other words, the exceedence flows are directly proportional to the mean flow rate. The deviation of the individual data points represents variations in characteristics of the sewer systems.

3.2 Diurnal Flow Variations

Daily flow charts were available from several plants to evaluate diurnal flow variation characteristics. Charts were selected for days on which the flows corresponded to the maximum day, 10 and 25 percent exceedence levels tabulated in the previous section. These gave a wide distribution of flow conditions for each system.

The charts were then analyzed to determine the flow rate occurring over several peak periods. The peak periods selected were the instantaneous peak, 1-hour peak, 4-hour peak, 6-hour peak and 12-hour peak. Peak flow rates were normalized with respect to the corresponding daily rates. The peaking factors were obtained by integration of the instantaneous flow rate over the specified time period and dividing by the average flows.

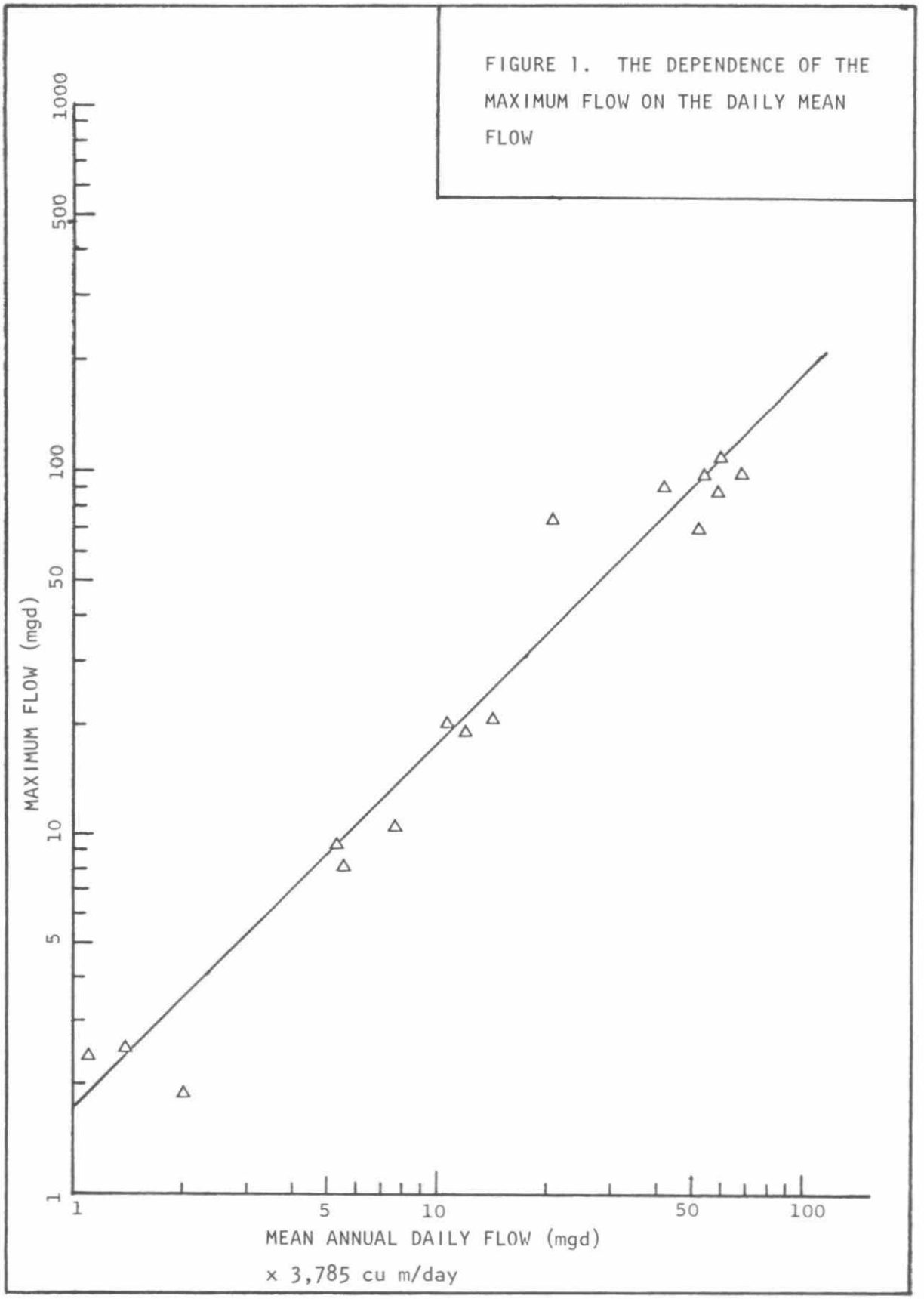


FIGURE 1. THE DEPENDENCE OF THE MAXIMUM FLOW ON THE DAILY MEAN FLOW

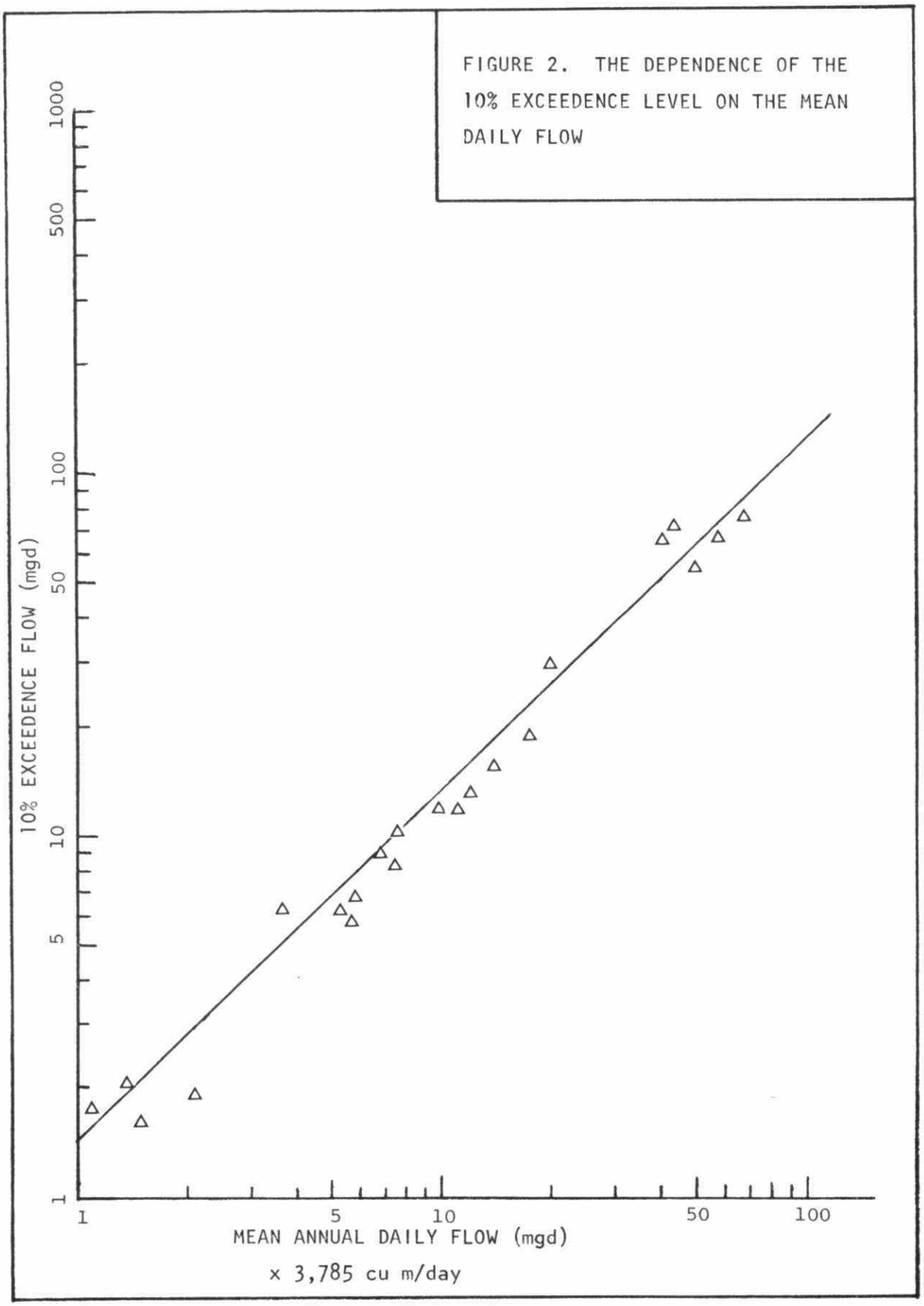


FIGURE 2. THE DEPENDENCE OF THE 10% EXCEEDENCE LEVEL ON THE MEAN DAILY FLOW

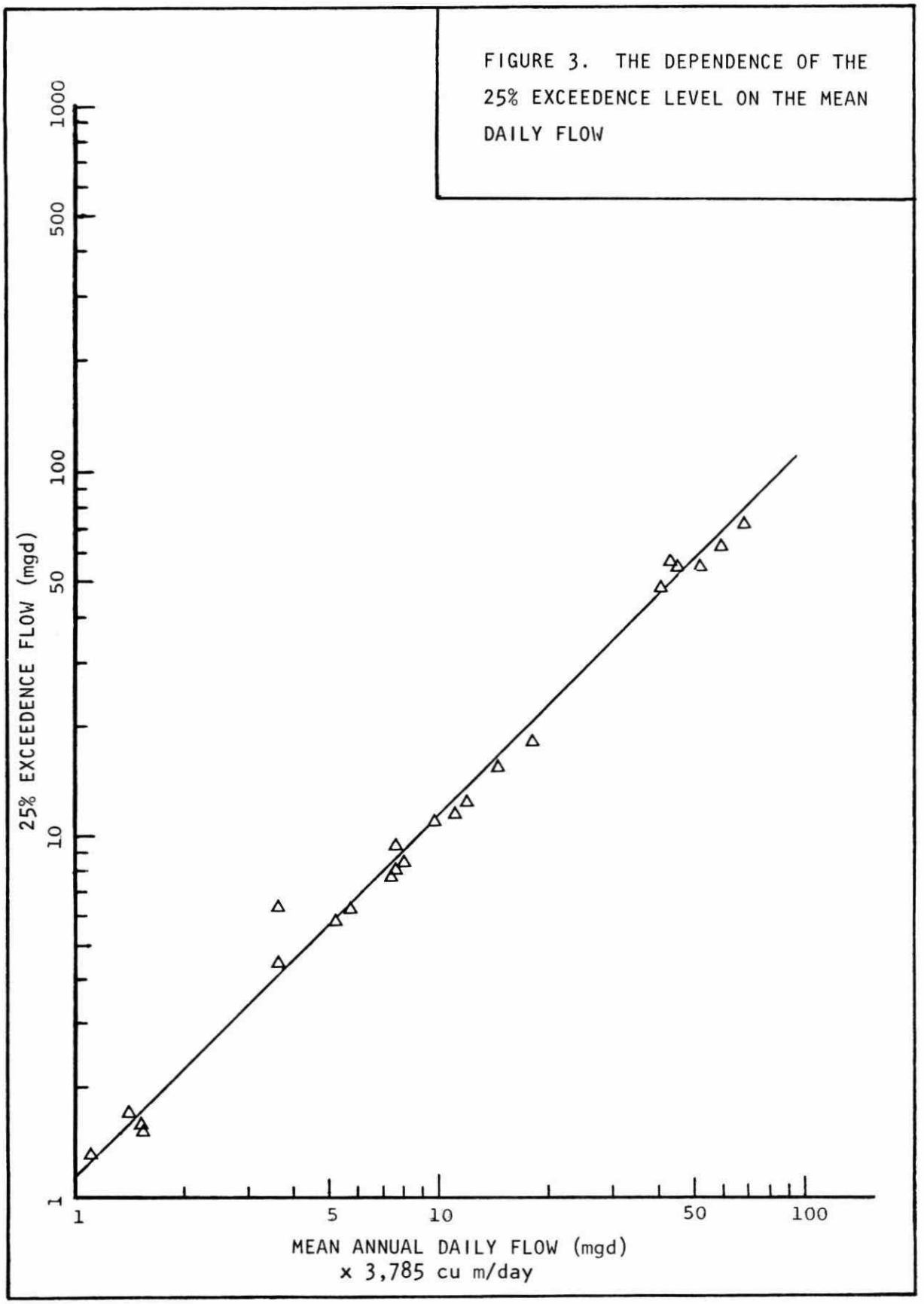


FIGURE 3. THE DEPENDENCE OF THE 25% EXCEEDENCE LEVEL ON THE MEAN DAILY FLOW

The diurnal peaking factors (PFd) are listed in Tables 4, 5, 6, and 7 under each of the five peaking periods studied for the selected exceedence levels. For a specific flow (e.g. Table 5) it is apparent that the PFd does, in general, decrease with increasing peaking periods. The change in PFd between any peaking period varies considerably from system to system and year to year.

3.2.1 The change in diurnal peaking factors (PFd) with dry and wet weather flows

The average diurnal peaking factors (PFd) are tabulated in Table 8. Generally, the diurnal peaking factors decrease with increasing daily mean flow. Thus, maximum diurnal flow variation tends to occur when the infiltration and storm water components of the sewage are at minimum value. As expected, the diurnal peaking factors decrease with increasing peaking periods. Although short-term peaking factors have no significance in the determination of the equalization basin volume, they are essential for establishing the pumping and the flow control requirements to and from the basin. The daily and the 12-hour peaking factors serve as key parameters for the realistic design of the equalization basin.

3.2.2 Variation in diurnal peaking factor (PFd) relative to sewage flow

Figures 4 to 23 show that there is a power function relationship between the flow for various peaking periods and the average daily flow at each exceedence level. That is, the equation, $y = a\ x^{n}$ describes the behaviour of the systems. The implications concerning the slope and the intercept are similar to those found previously for the daily peaking factors (PF).

Table 9 shows that the slopes of the log-linear relationships are essentially identical. The intercepts do not vary noticeably with respect to increasing or decreasing daily flows. However, the intercepts do decrease slightly with increasing peaking period.

From Tables 4, 5, 6, and 7, it was observed that the PFd varied randomly within a narrow range and was independent of annual average flow. As mentioned in Section 3.2 some of the data reflect specific local system characteristics. For example, systems B and H incorporate pumping of raw sewage, and this therefore results in step increases in the diurnal

TABLE 4. DIURNAL PEAKING FACTORS (PFd) DURING MAXIMUM FLOW CONDITIONS

Municipality	Year	Annual* Mean Flow (mgd)	Peak Flow	1-Hr	4-Hr	6-Hr	12-Hr
A	1971	7.7	1.22	1.20	1.17	1.17	1.12
H	1968	40.09	1.11	1.10	1.08	1.08	1.06
	1969	44.42	1.06	1.06	1.05	1.04	1.03
	1970	44.98	1.01	1.00	1.00	1.00	1.00
I	1971	5.21	1.51	1.51	1.48	1.48	1.44
J	1972	20.5	1.27	1.24	1.19	1.17	1.07
	1973	26.8	1.47	1.46	1.38	1.36	1.24

*x 3,785 cu m/day

TABLE 5. DIURNAL PEAKING FACTORS DURING 10% EXCEEDENCE FLOW CONDITIONS

Municipality	Year	Mean Flow (mgd)*	Peak Flow	1-Hr	4-Hr	6-Hr	12-Hr
A	1971	7.7	1.24	1.24	1.22	1.21	1.15
	1972	10.7	1.29	1.20	1.18	1.15	1.11
B	1972	7.62	1.04	1.03	-	-	-
	1973	9.66	1.11	1.11	1.09	1.09	-
D	1972	7.28	1.16	1.15	1.15	1.14	1.12
	1973	7.25	1.25	1.24	1.22	1.21	1.17
G	1971	1.12	1.24	1.12	1.12	1.11	1.09
H	1968	40.09	1.22	1.10	1.08	1.05	1.00
	1969	44.42	1.17	1.16	1.12	1.09	1.09
	1970	44.98	1.14	1.14	1.14	1.12	-
I	1971	5.21	1.32	1.21	1.20	1.19	1.17
	1972	5.55	1.34	1.22	1.21	1.14	1.15
J	1972	20.5	1.17	1.13	1.12	1.11	1.08
	1973	20.8	1.09	1.09	1.08	1.07	1.03

*x 3,785 cu m/day

TABLE 6. DIURNAL PEAKING FACTORS DURING 25% EXCEEDENCE FLOW CONDITIONS

Municipality	Year	Mean Flow (mgd)*	Peak Flow	1-Hr	4-Hr	6-Hr	12-Hr
A	1971	7.70	1.28	1.27	1.26	1.25	1.21
	1972	10.73	1.37	1.21	1.28	1.16	1.12
D	1972	7.28	1.26	1.25	1.23	1.22	1.17
	1973	7.09	1.33	1.31	1.28	1.25	1.20
G	1971	1.12	1.35	1.25	1.20	1.17	1.15
	1972	1.41	1.32	1.20	1.20	1.14	1.14
H	1968	40.09	1.32	1.30	1.28	1.26	1.18
	1969	44.42	1.16	1.16	1.15	1.14	1.12
	1970	44.98	1.15	1.15	1.12	1.12	-
I	1971	5.21	1.39	1.29	1.28	1.22	1.15
	1972	5.55	1.29	1.21	1.21	1.20	1.15
J	1972	20.50	1.41	1.41	1.39	1.31	1.26
	1973	20.80	1.16	1.16	1.14	1.14	1.09

*x 3,785 cu m/day

TABLE 7. DIURNAL PEAKING FACTORS DURING MEAN DAILY FLOW CONDITIONS

Municipality	Year	Mean Flow (mgd)*	Peak Flow	1-Hr	4-Hr	6-Hr	12-Hr
A	1971	7.70	1.33	1.31	1.29	1.26	1.18
	1972	10.73	1.36	1.22	1.21	1.21	1.16
B	1971	5.49	1.23	1.22	1.21	1.20	1.19
	1972	7.62	1.19	1.19	1.11	1.10	1.11
D	1973	7.09	1.26	1.25	1.24	1.23	1.19
G	1971	1.12	1.51	1.39	1.29	1.18	1.18
	1972	1.41	1.39	1.29	1.26	1.23	1.19
H	1968	40.09	1.28	1.28	1.25	1.26	1.22
	1969	44.42	1.26	1.23	1.21	1.20	1.12
	1970	44.98	1.33	1.31	1.29	1.27	1.18
I	1971	5.21	1.12	1.08	1.06	1.05	1.05
	1972	5.55	1.34	1.20	1.20	1.17	1.15
J	1972	20.5	1.26	1.26	1.24	1.24	1.17
	1973	20.8	1.16	1.14	1.14	1.12	1.09

*x 3,785 cu m/day

TABLE 8. AVERAGE DIURNAL PEAKING FACTORS FOR VARIOUS PEAKING PERIODS

		Peak	1-Hr	4-Hr	6-Hr	12-Hr
Maximum Daily Flow	Average	1.23	1.22	1.19	1.18	1.13
	S.D.*	0.17	0.17	0.16	0.15	0.14
10% Exceedence Flow	Average	1.20	1.15	1.15	1.14	1.14
	S.D.	0.077	0.062	0.055	0.040	0.055
25% Exceedence Flow	Average	1.28	1.24	1.23	1.20	1.16
	S.D.	0.084	0.057	0.071	0.046	0.045
Mean Daily Flow	Average	1.29	1.24	1.21	1.19	1.16
	S.D.	0.10	0.077	0.071	0.074	0.045

*S.D. = Standard Deviation

TABLE 9. THE DEPENDENCE OF THE CHARACTERISTIC CONSTANTS OF DIURNAL PEAK FLOWS ON THE PEAKING PERIOD

Category	a	n	Category	a	n
Maximum Day			25% Exceedence Flow		
Instant Peak Flow	1.457	0.940	Peak Flow	1.37	0.956
1-hr Peak	1.336	0.961	1-hr Peak	1.30	0.967
4-hr Peak	1.412	0.940	4-hr Peak	1.28	0.985
6-hr Peak	1.382	0.946	6-hr Peak	1.20	0.988
12-hr Peak	1.255	0.980	12-hr Peak	1.19	0.981
10% Exceedence Flow			Mean Flow		
Peak Flow	1.194	0.995	Peak Flow	1.38	0.977
1-hr Peak	1.169	0.989	1-hr Peak	1.32	0.969
4-hr Peak	1.14	0.996	4-hr Peak	1.25	0.982
6-hr Peak	1.21	0.964	6-hr Peak	1.19	1.00
12-hr Peak	1.12	0.978	12-hr Peak	1.19	0.931

flow patterns. The diurnal flow charts for system C showed almost constant flows due to a high and continuous infiltration.

3.3 Combination of Daily and Diurnal Peaking Factors

The daily and diurnal peaking factors conveniently represent variations in hydraulic loadings to a sewage treatment plant. The daily peaking factor (PF) indicates variations in daily flow from the mean annual daily flow. The diurnal peaking factor (PFd) represents fluctuations in average flow for various peaking periods.

Multiplying these two peaking factors and a specified daily flow it is possible to estimate the flow for any peaking period. The procedure for estimating peak design flows, therefore, is simplified into selecting appropriate peaking factors.

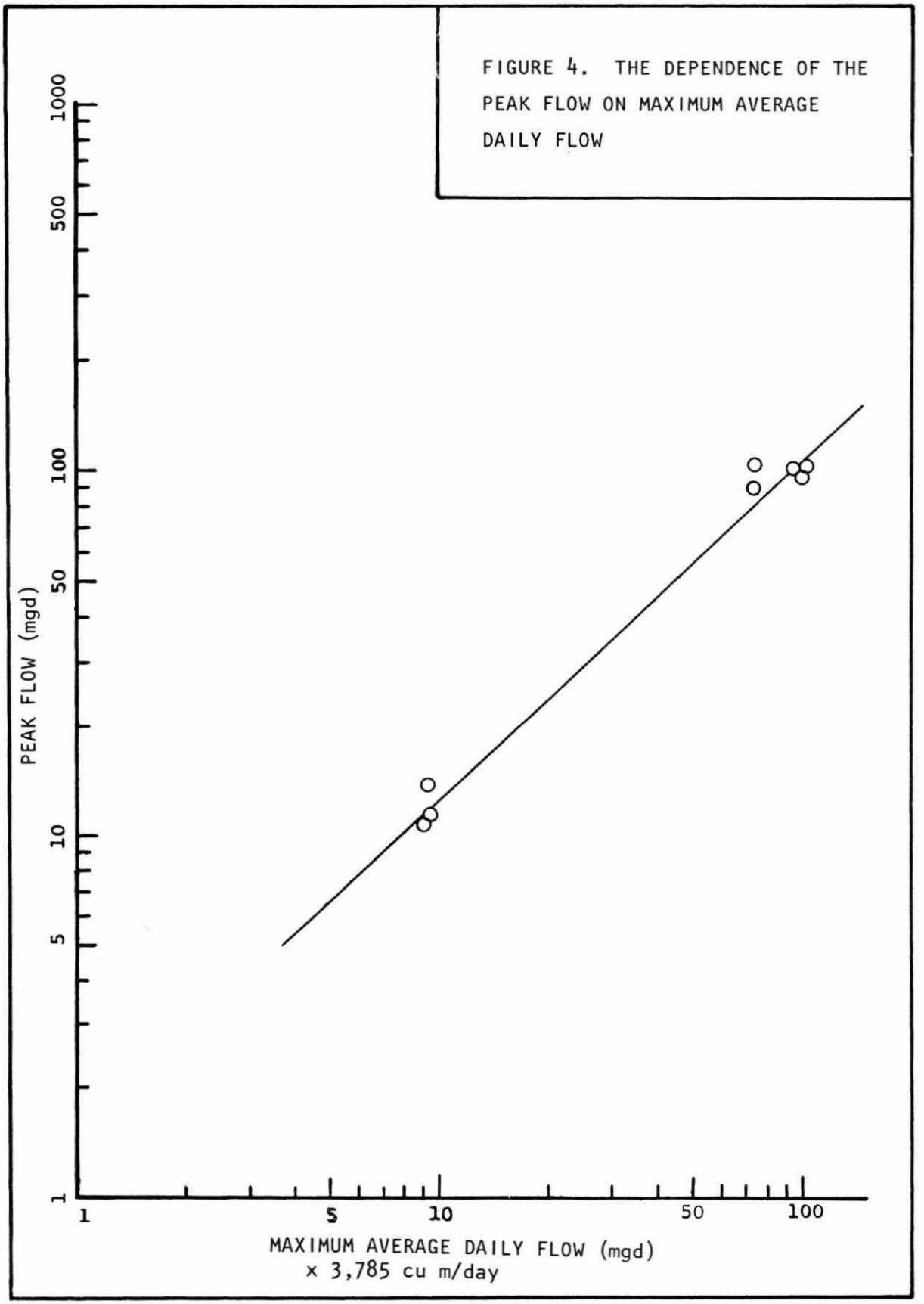


FIGURE 4. THE DEPENDENCE OF THE PEAK FLOW ON MAXIMUM AVERAGE DAILY FLOW

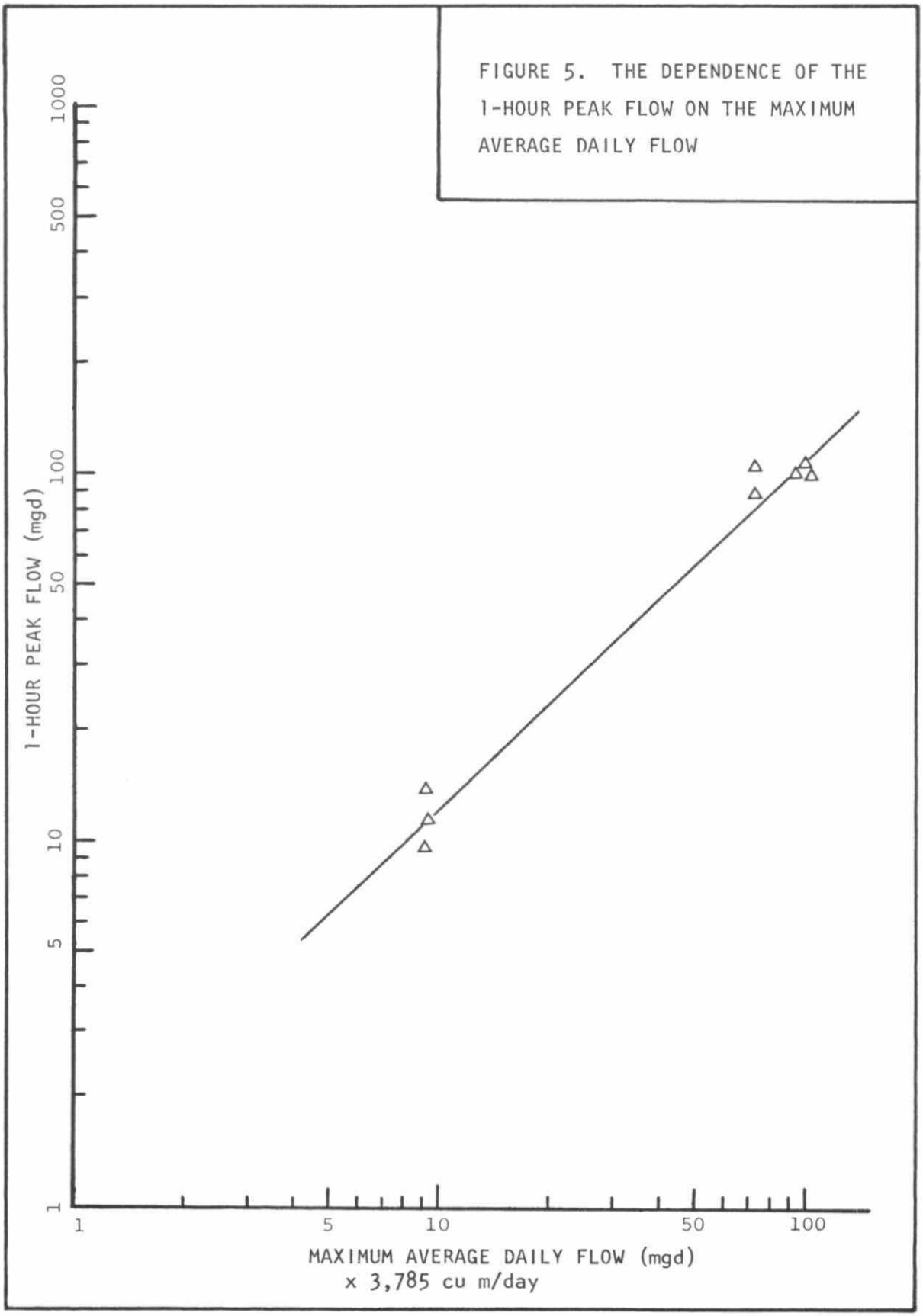


FIGURE 5. THE DEPENDENCE OF THE 1-HOUR PEAK FLOW ON THE MAXIMUM AVERAGE DAILY FLOW

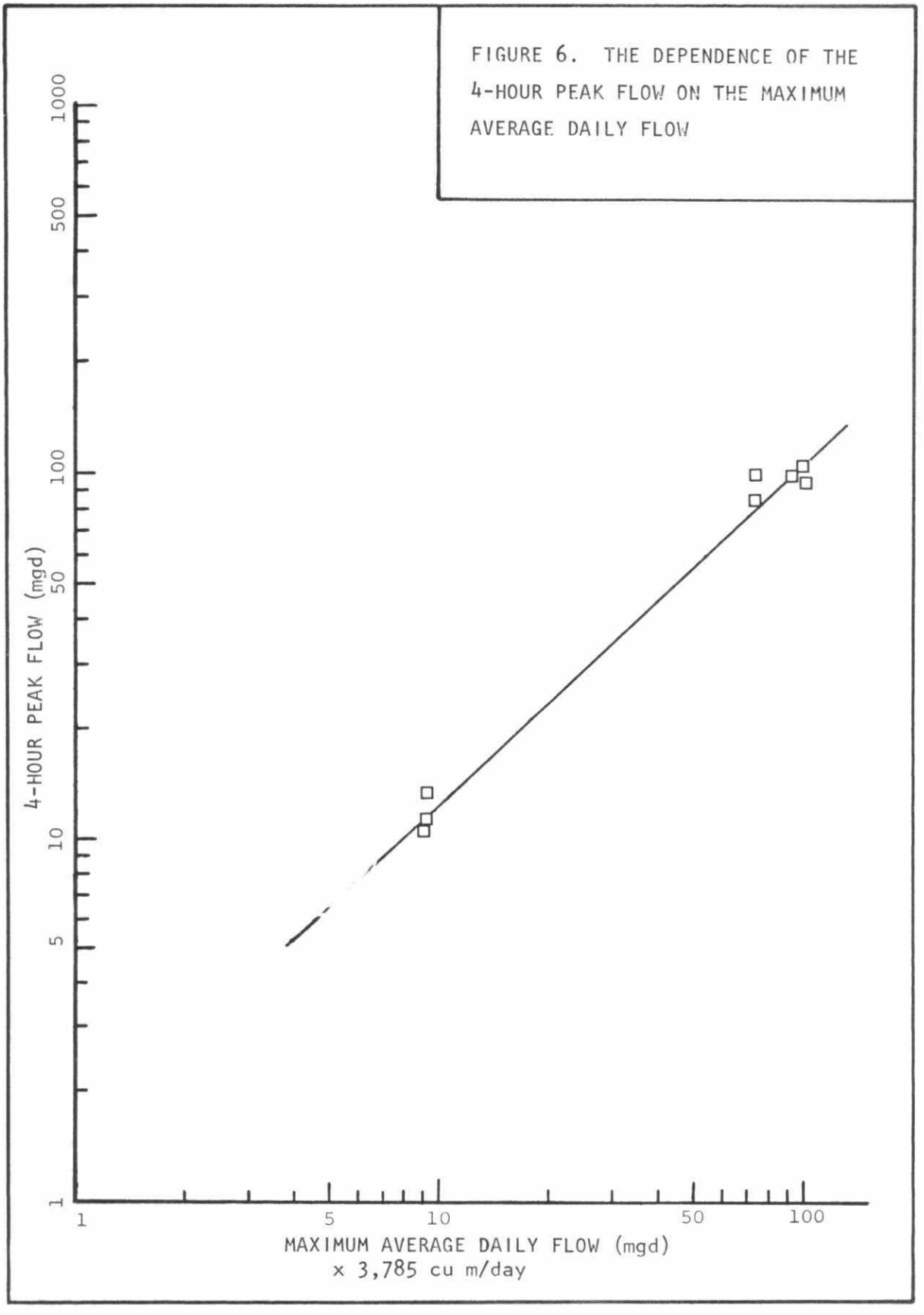


FIGURE 6. THE DEPENDENCE OF THE 4-HOUR PEAK FLOW ON THE MAXIMUM AVERAGE DAILY FLOW

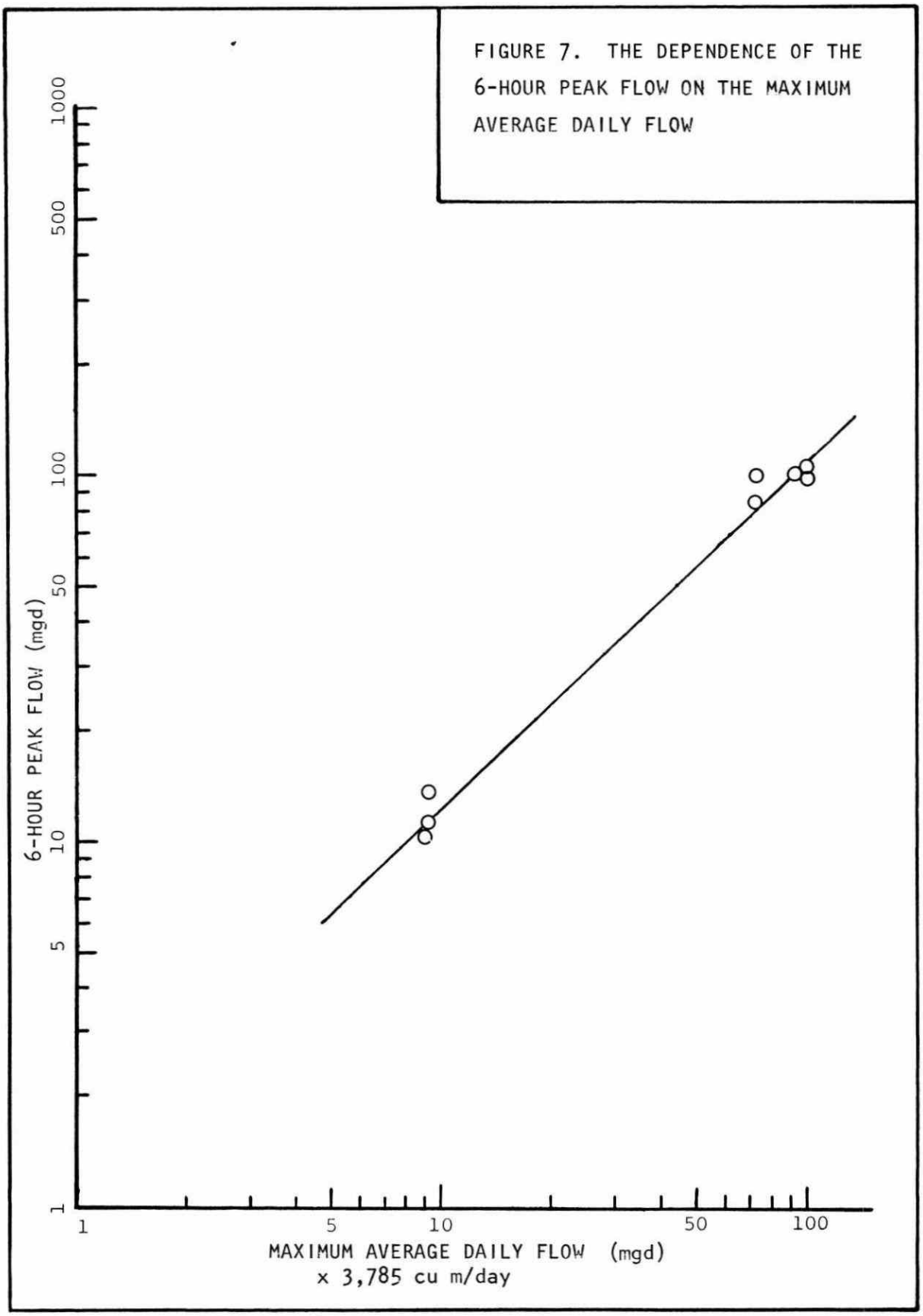


FIGURE 7. THE DEPENDENCE OF THE 6-HOUR PEAK FLOW ON THE MAXIMUM AVERAGE DAILY FLOW

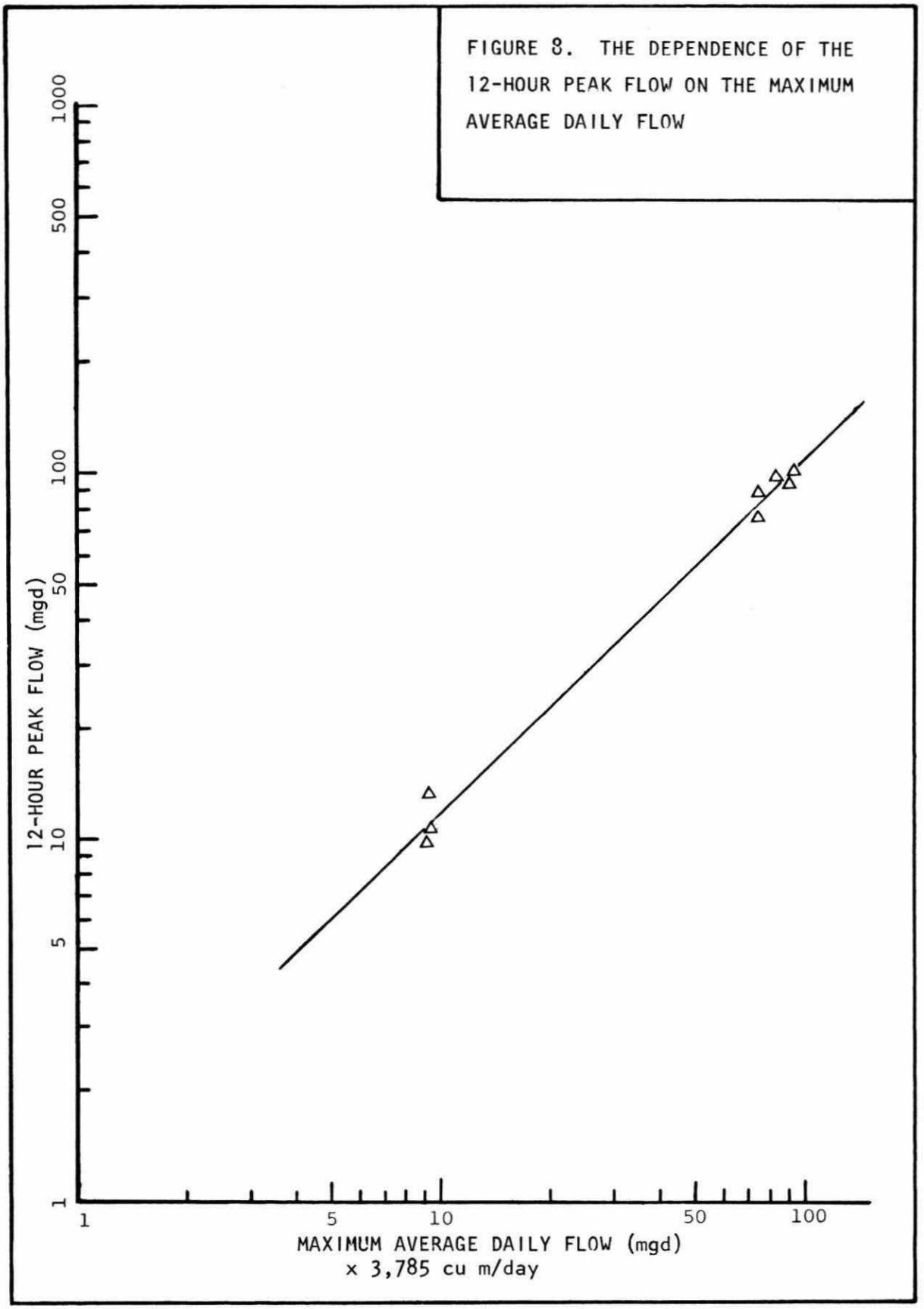


FIGURE 8. THE DEPENDENCE OF THE 12-HOUR PEAK FLOW ON THE MAXIMUM AVERAGE DAILY FLOW

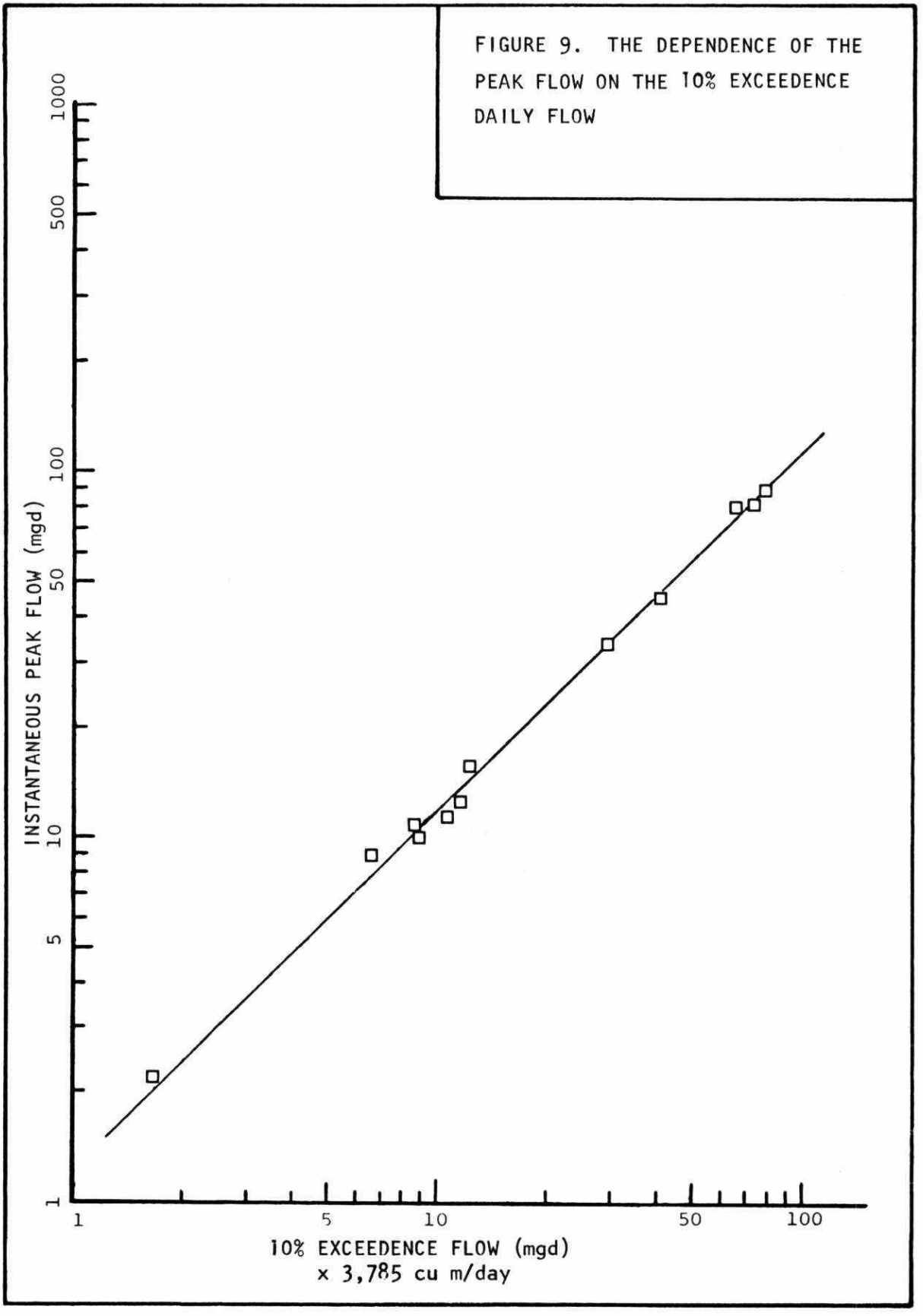
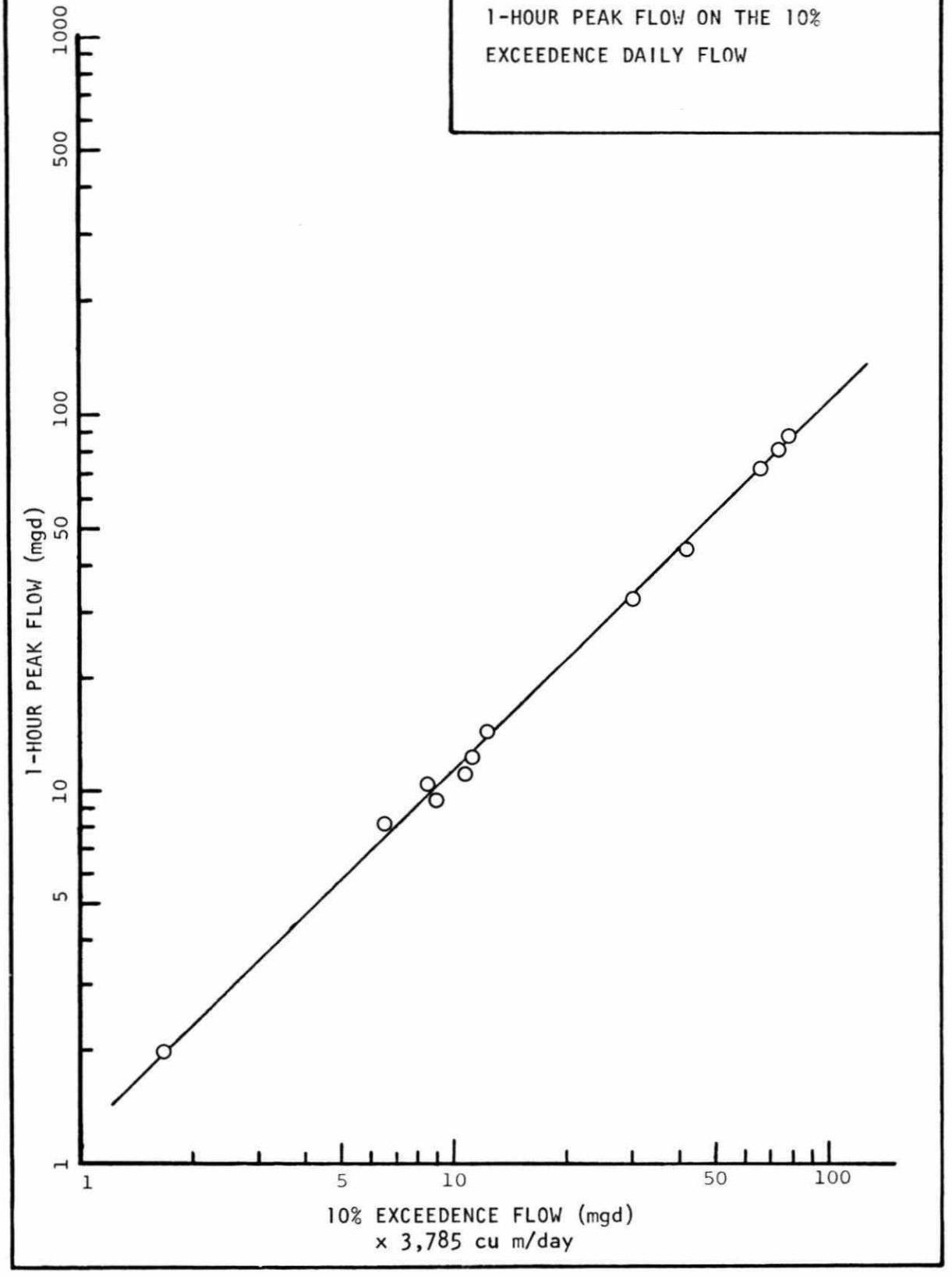


FIGURE 9. THE DEPENDENCE OF THE PEAK FLOW ON THE 10% EXCEEDENCE DAILY FLOW

FIGURE 10. THE DEPENDENCE OF THE 1-HOUR PEAK FLOW ON THE 10% EXCEEDENCE DAILY FLOW



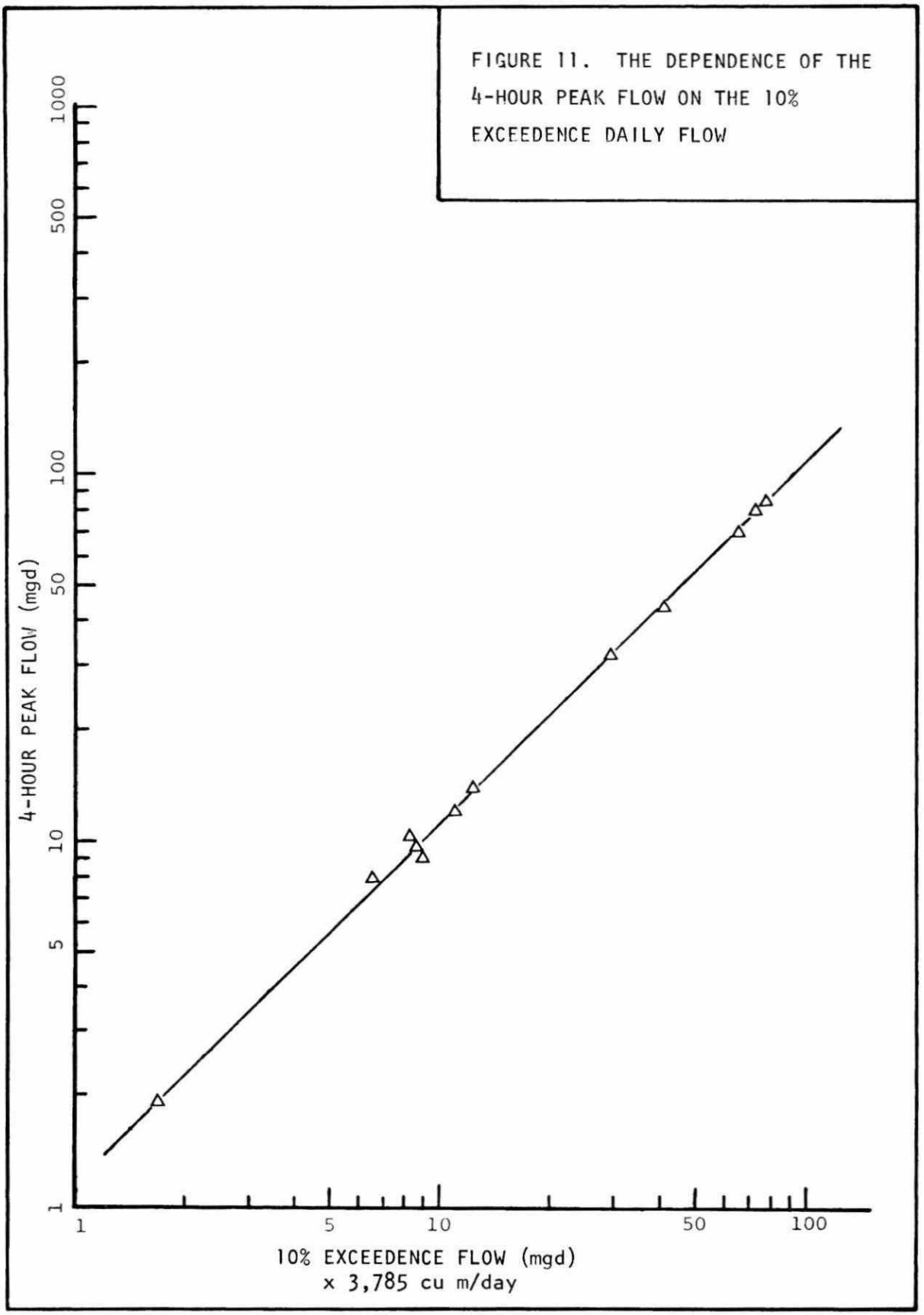
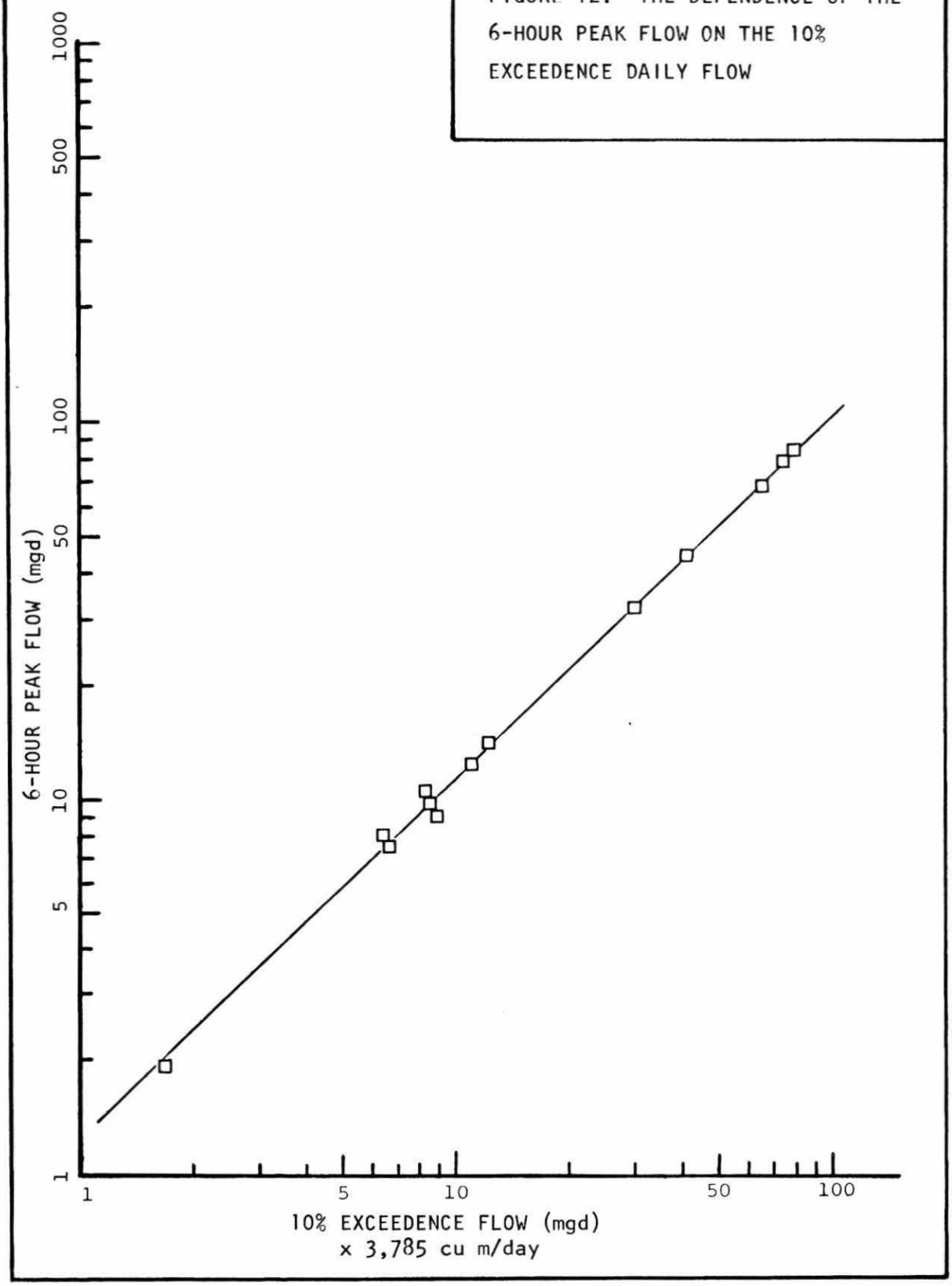


FIGURE 11. THE DEPENDENCE OF THE 4-HOUR PEAK FLOW ON THE 10% EXCEEDENCE DAILY FLOW

FIGURE 12. THE DEPENDENCE OF THE 6-HOUR PEAK FLOW ON THE 10% EXCEEDENCE DAILY FLOW



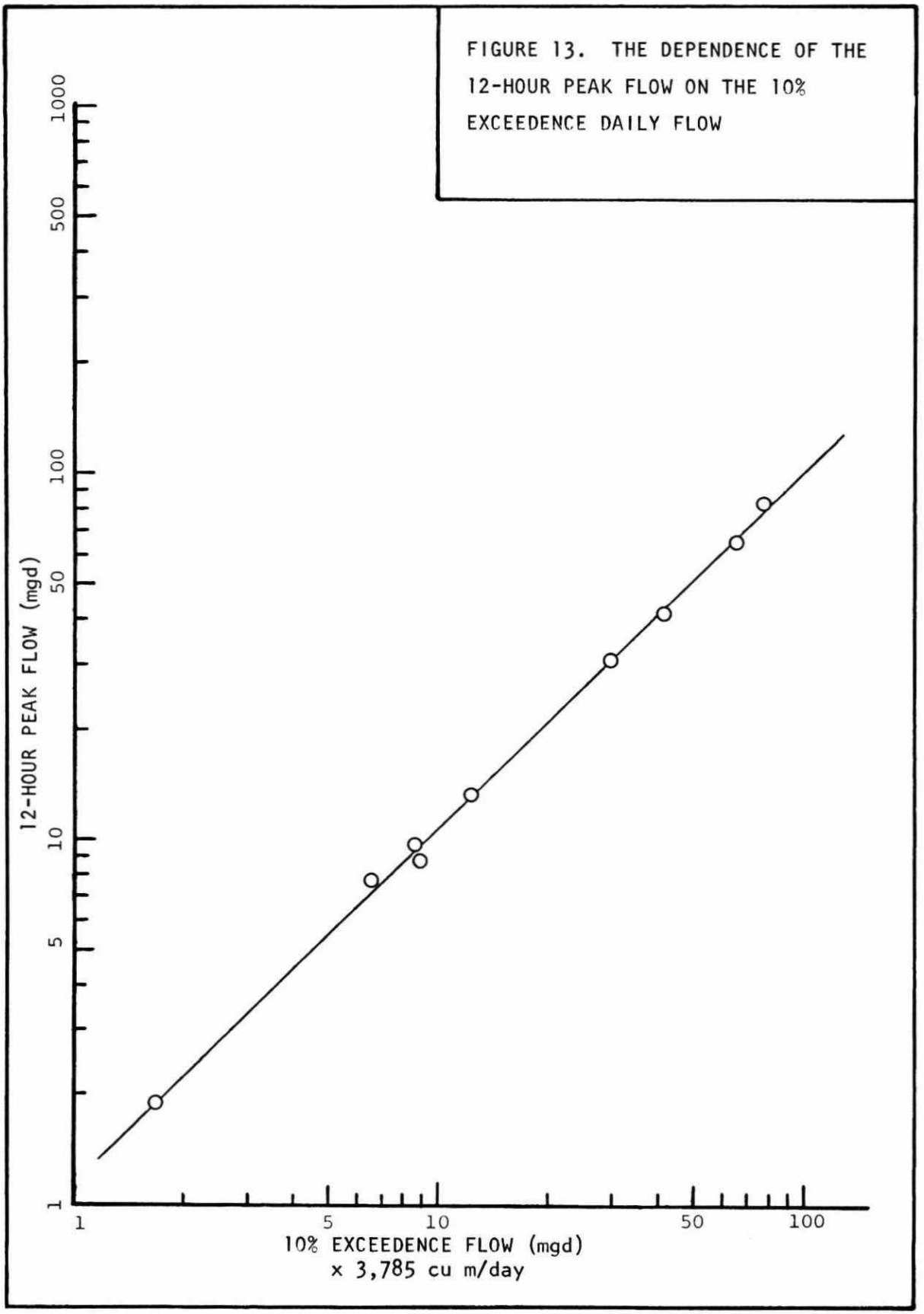


FIGURE 13. THE DEPENDENCE OF THE 12-HOUR PEAK FLOW ON THE 10% EXCEEDENCE DAILY FLOW

FIGURE 14. THE DEPENDENCE OF THE MAXIMUM FLOW ON THE 25% EXCEEDENCE DAILY FLOW

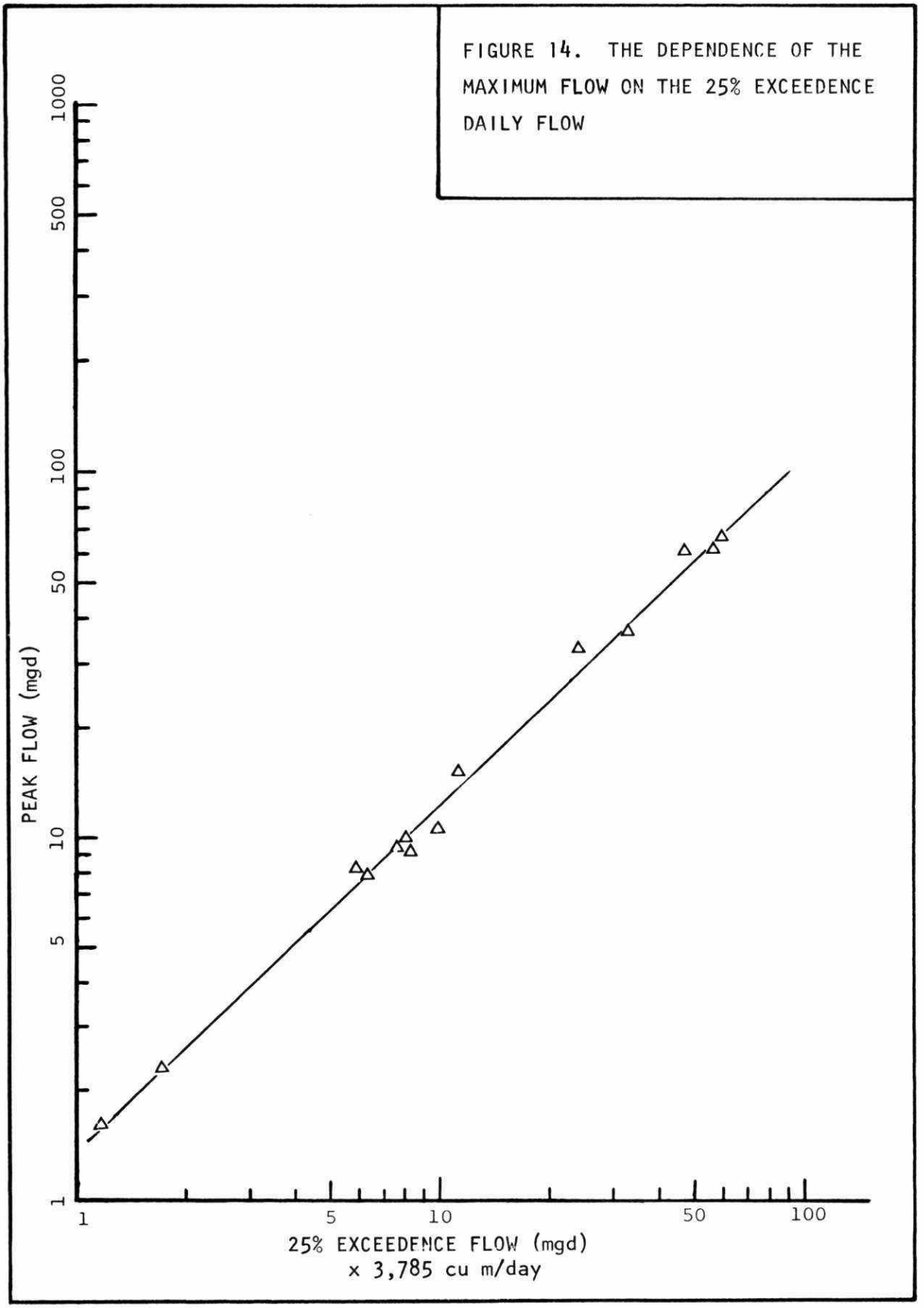


FIGURE 15. THE DEPENDENCE OF THE 1-HOUR PEAK FLOW ON THE 25% EXCEEDENCE DAILY FLOW

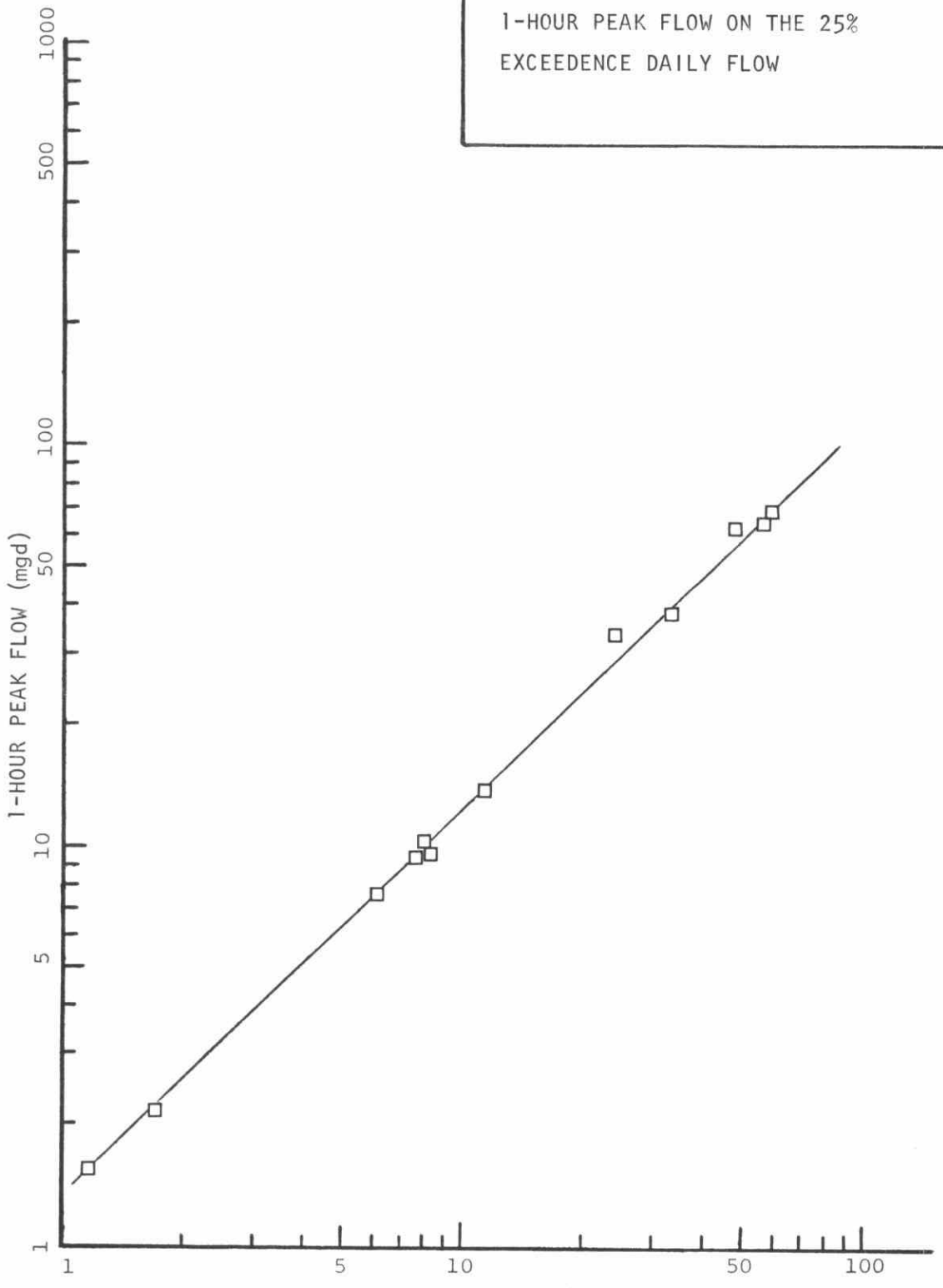
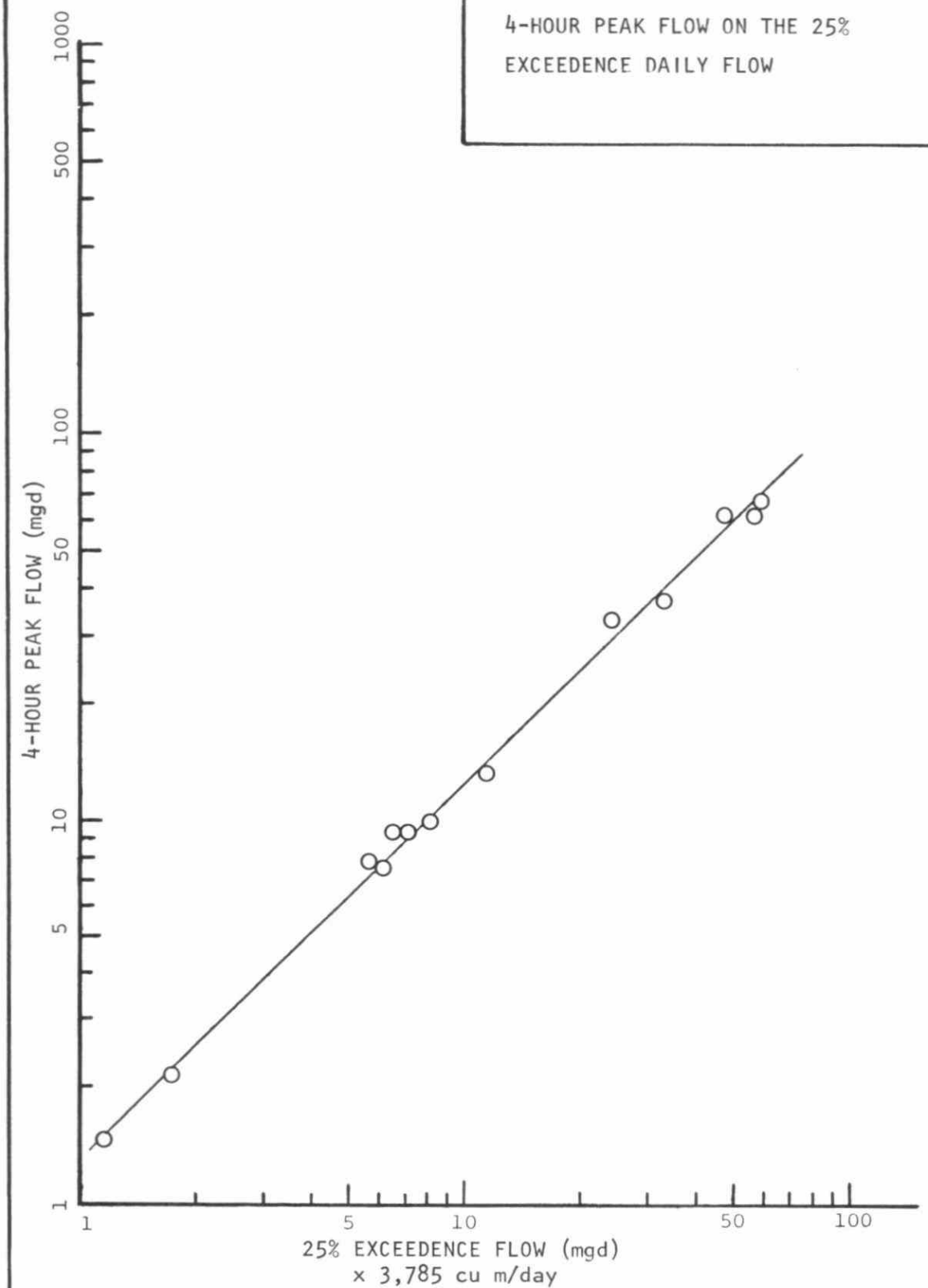


FIGURE 16. THE DEPENDENCE OF THE 4-HOUR PEAK FLOW ON THE 25% EXCEEDENCE DAILY FLOW



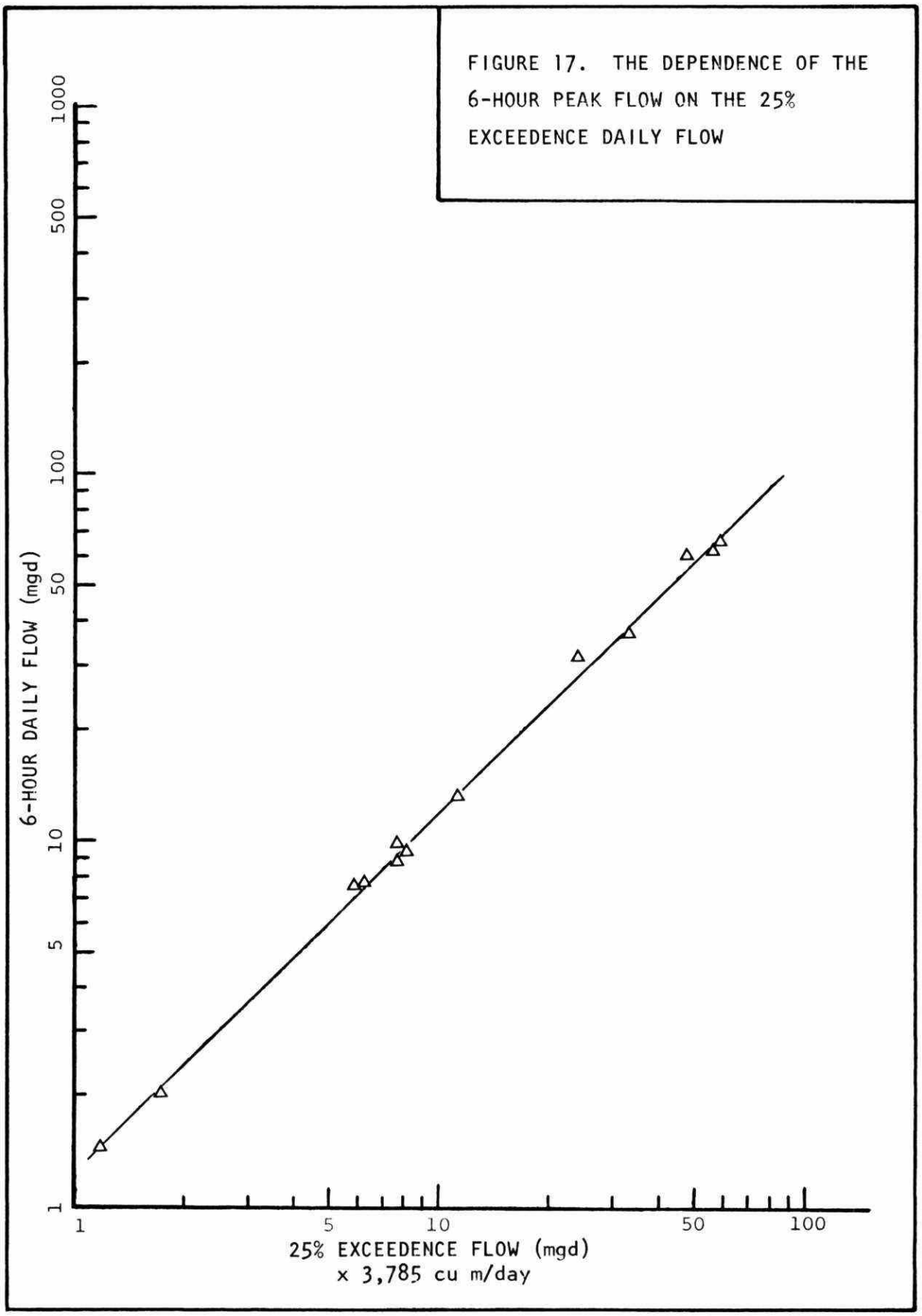
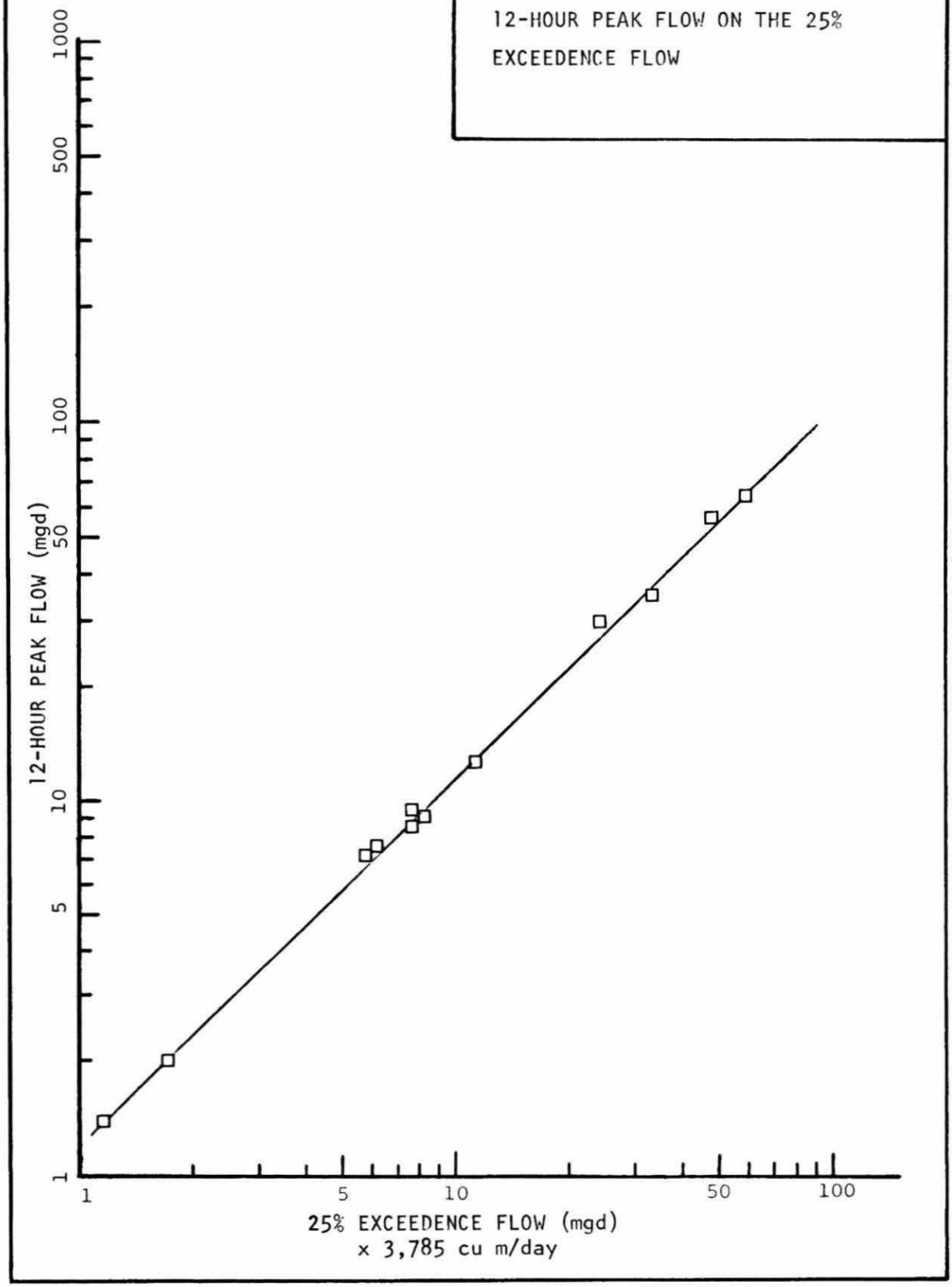


FIGURE 17. THE DEPENDENCE OF THE 6-HOUR PEAK FLOW ON THE 25% EXCEEDENCE DAILY FLOW

FIGURE 18. THE DEPENDENCE OF THE 12-HOUR PEAK FLOW ON THE 25% EXCEEDENCE FLOW



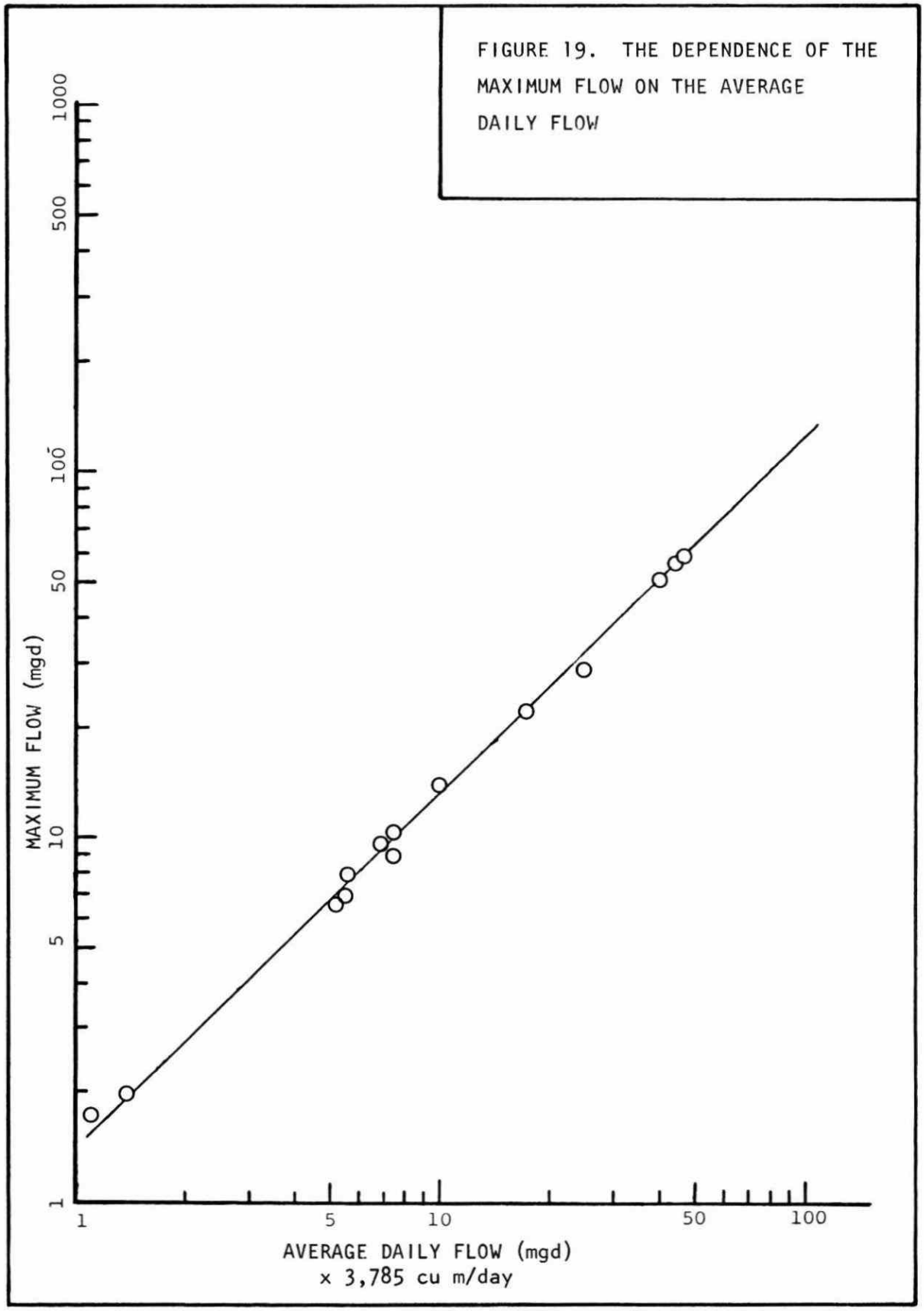
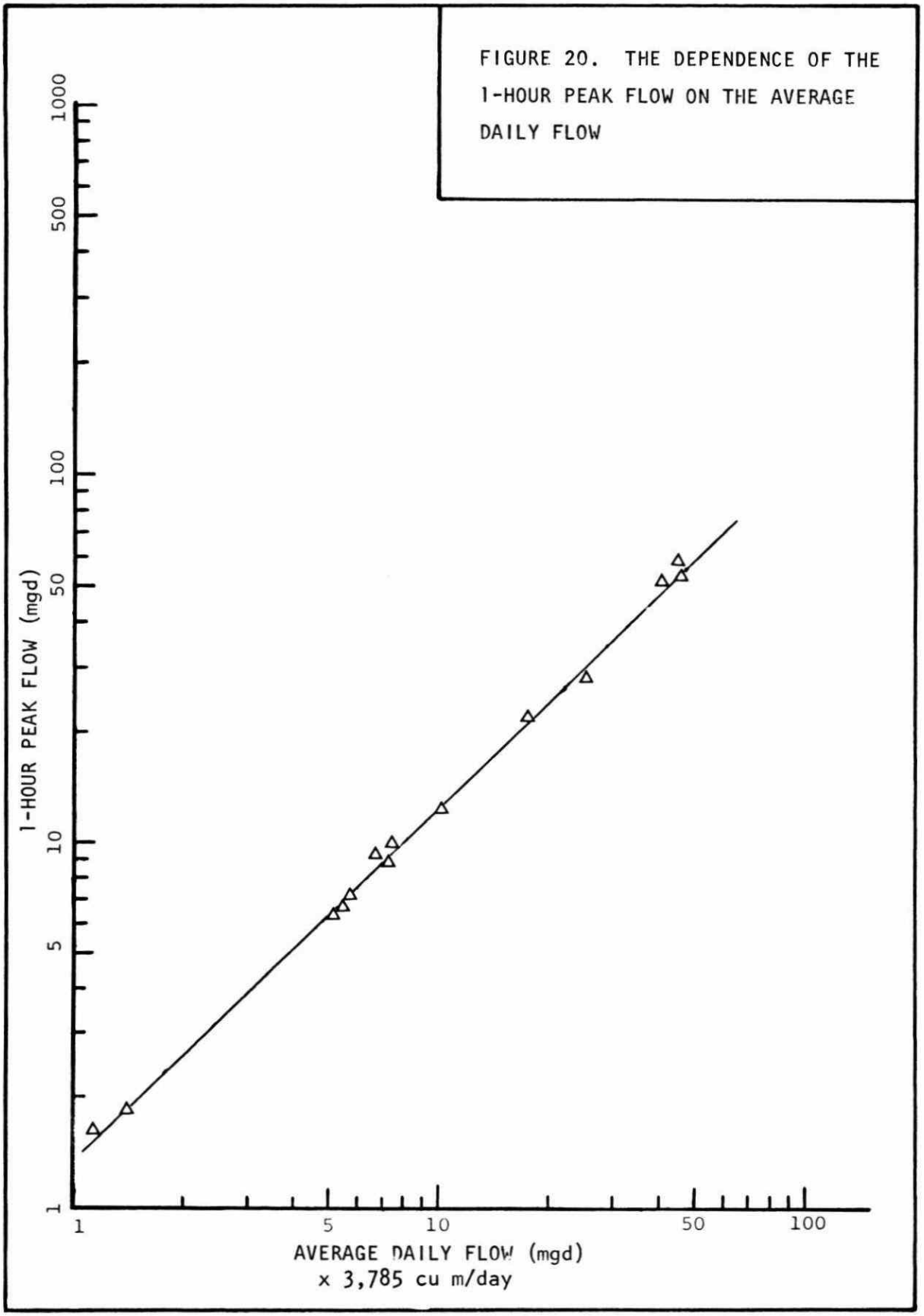


FIGURE 19. THE DEPENDENCE OF THE MAXIMUM FLOW ON THE AVERAGE DAILY FLOW

FIGURE 20. THE DEPENDENCE OF THE 1-HOUR PEAK FLOW ON THE AVERAGE DAILY FLOW



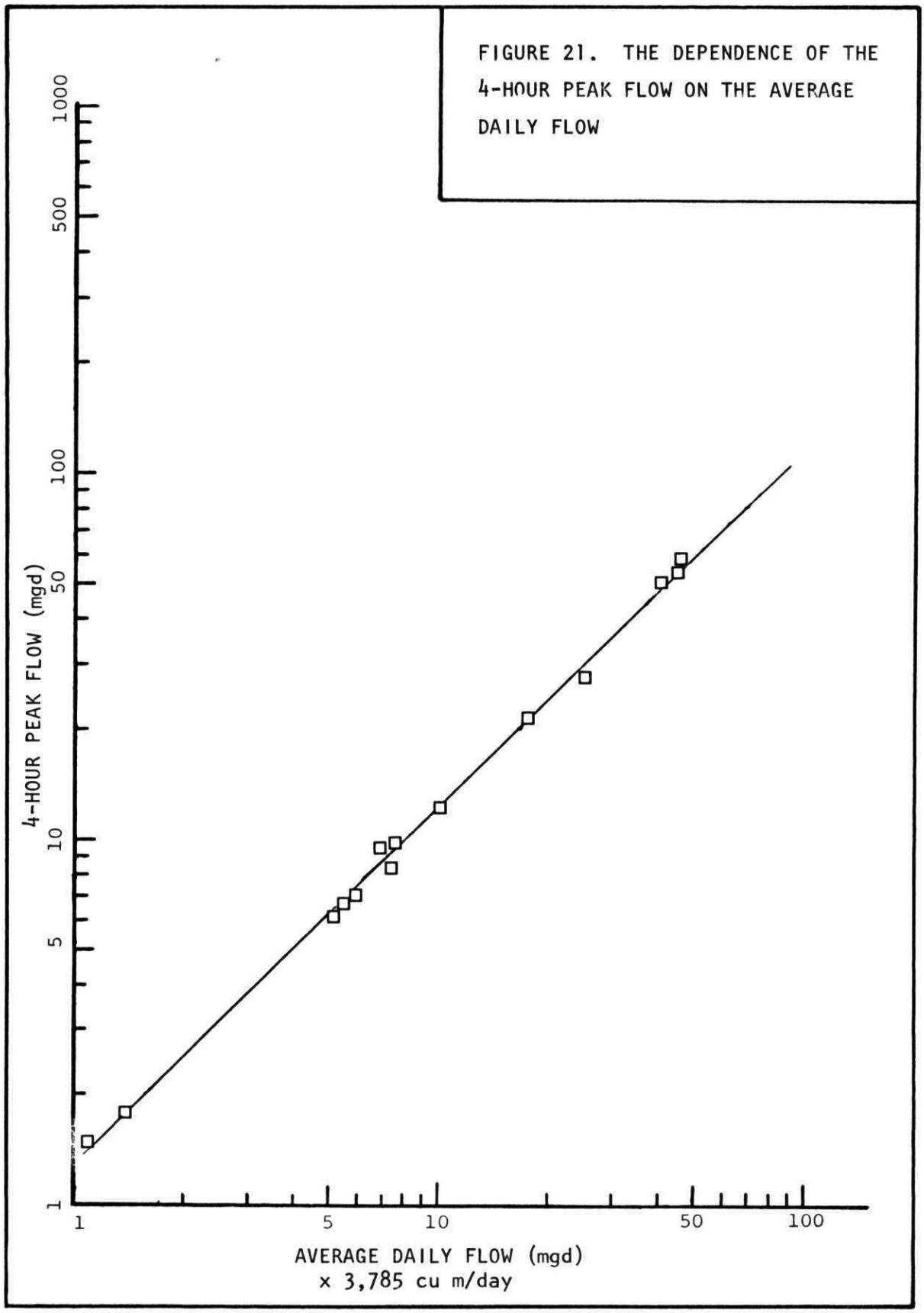
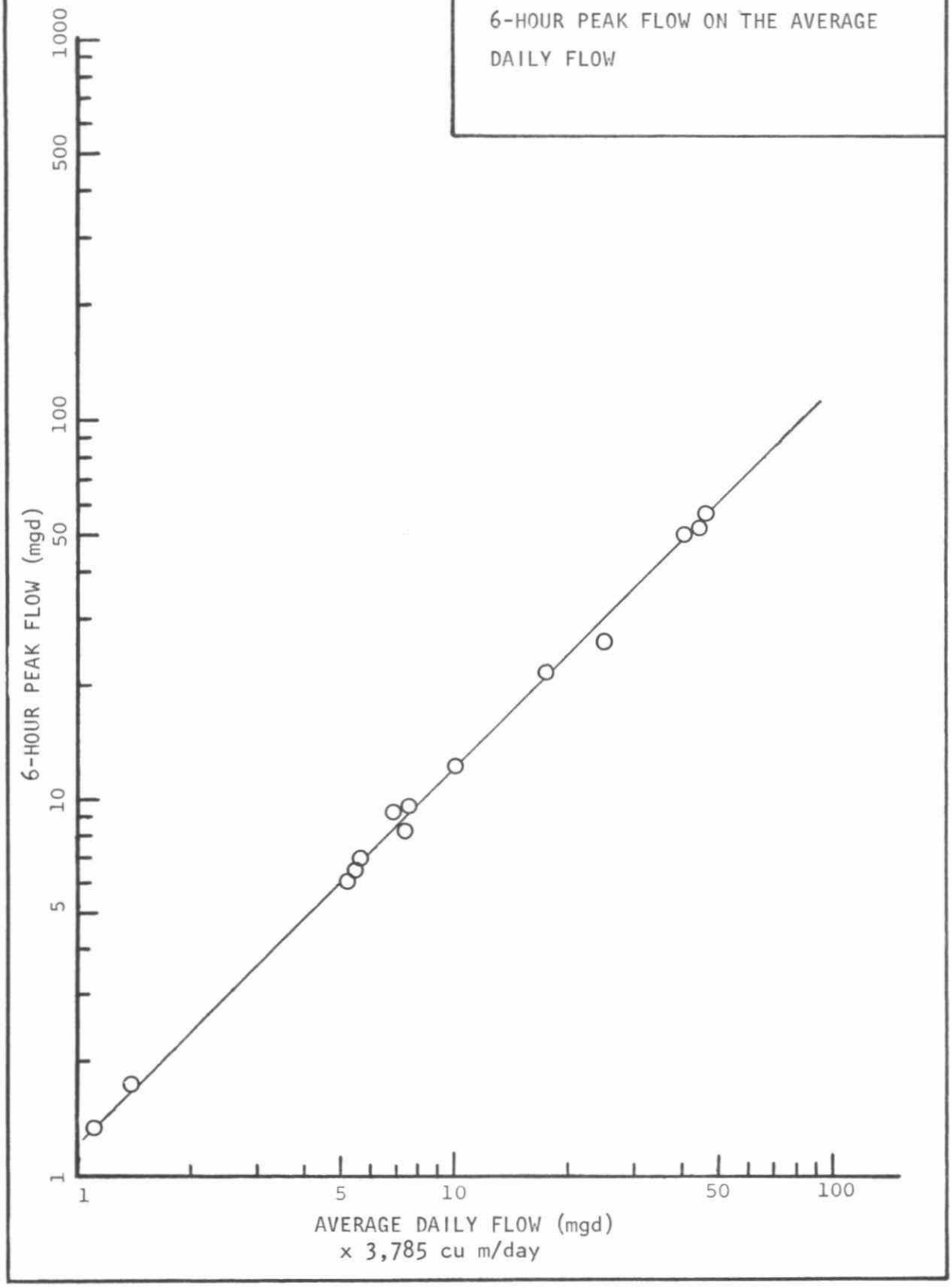


FIGURE 21. THE DEPENDENCE OF THE 4-HOUR PEAK FLOW ON THE AVERAGE DAILY FLOW

FIGURE 22. THE DEPENDENCE OF THE 6-HOUR PEAK FLOW ON THE AVERAGE DAILY FLOW



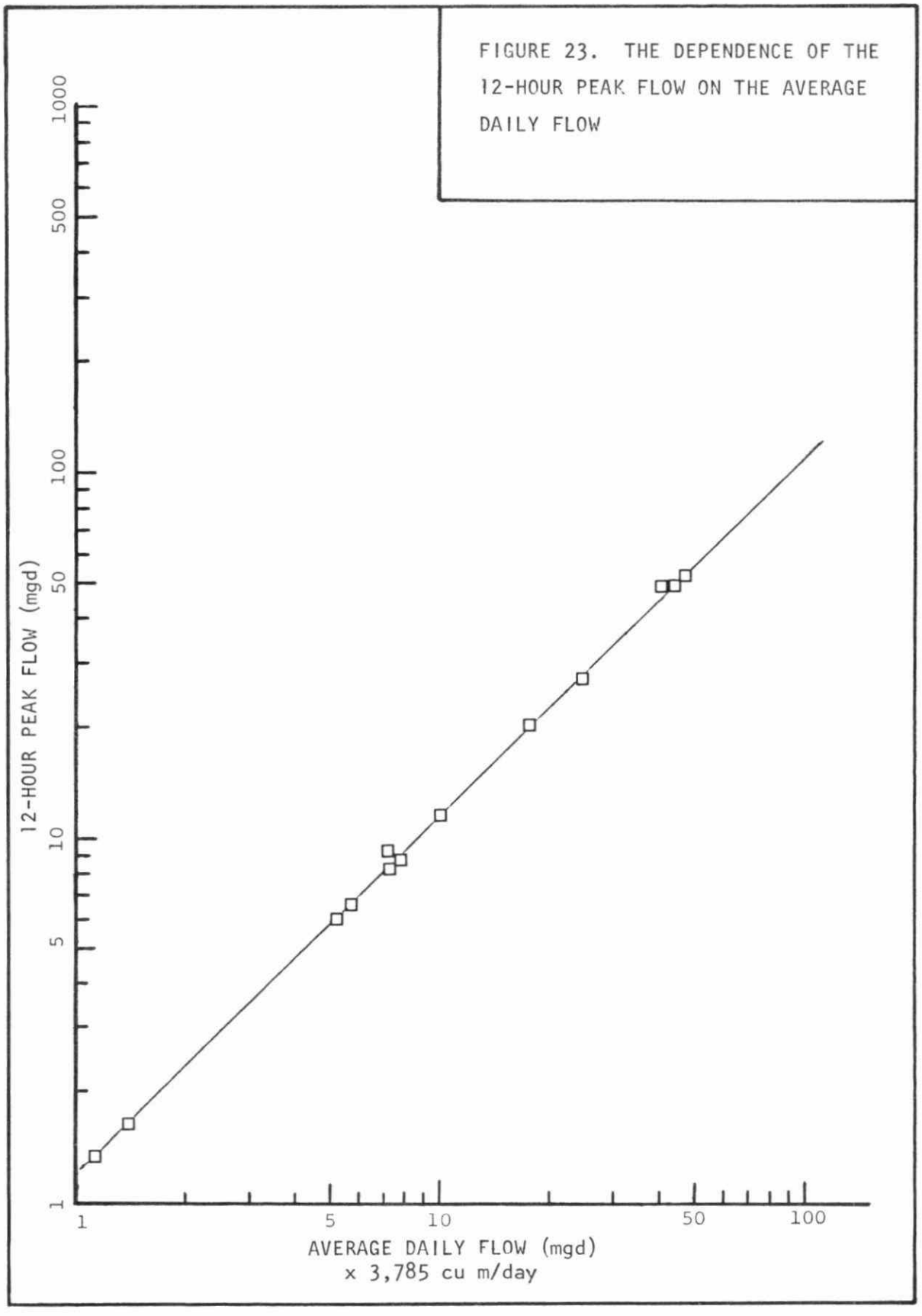


FIGURE 23. THE DEPENDENCE OF THE 12-HOUR PEAK FLOW ON THE AVERAGE DAILY FLOW

4. COMPARISON OF DESIGN CRITERIA AND COST OF WPCP

4.1 Design Criteria

In the following design examples the daily and diurnal peaking factors are used. The design basis for treatment processes are discussed relative to variable flow and equalized flow systems. Subsequently, the cost of four sewage treatment plant design alternatives are examined. The alternatives include designing for full, partial and no equalization without bypassing at the treatment plant. The cost estimations are performed for sewage treatment plants with 5 mgd (19,000 cu m/day) and 50 mgd (190,000 cu m/day) mean annual daily flow.

4.2 Sizing of Equalization Basins

In the past only very limited data have been published on the subject of equalization basin design for domestic sewage treatment plants. In a previous report [1] the maximum volume requiring flow equalization was estimated from diurnal flow data for a 50 mgd plant to be 11.5% of the maximum daily average flow. This is equivalent to 23% of mean annual daily flow for the 50 mgd plant studied.

LaGrega and Keenan [2] developed a computer prediction of diurnal flow variations assuming a sinusoidal pattern and a specific relationship between maximum peak diurnal flow and average daily flow as follows:

$$y = 1.78 \cdot x^{0.92}$$

where: y = the peak diurnal flow (mgd)

x = average daily flow (mgd).

The maximum flow volume requiring storage was determined by numerical integration, and was found to equal 12.5% of the mean annual daily flow for plants ranging in size from 1 to 100 mgd (3,800 to 380,000 cu m/day). Smith et al [3] used an empirical design volume of 15% of mean annual daily flow for plants of unspecified size.

Due to the stochastic nature of raw sewage flow, the sizing of equalization basins requires the forecasting of daily and diurnal flow profiles. In this report the fluctuations in flow were expressed as daily

and diurnal peaking factors (PF and PFd). The peaking factors, in turn, were used as key parameters for the realistic design of equalization basins.

The volume of any equalization basin is also dependent on the desired degree of equalization. In this report full equalization was based on the flow profile and corresponding peaking factors for maximum daily flow. Partial equalization was accomplished by taking into consideration the flow conditions at a particular flow exceedence level. The methodology for evaluating volume requirements for full and partial equalization was identical.

The basic concept involved in the development of the equation for flow equalization design is illustrated on Figure 24.

On this diagram an idealized instantaneous flow rate is plotted against time over one day. The instantaneous flow rate exceeds the daily mean during a time period, ΔT. This time period, defined as the peaking period, is the elapsed time between T_1 and T_2. If the flow were to be equalized for maintaining the average flow rate $\bar{Q}$, then the volume of raw sewage requiring storage is the time integral of the difference between the instantaneous flow rate (Q) and the average daily flow rate ($\bar{Q}$) over the peaking period (ΔT) i.e.:

$$V = \int_{T_1}^{T_1 + \Delta T} (Q - \bar{Q})dt = \int_{T_1}^{T_1 + \Delta T} Q\,dt - \bar{Q}(\Delta T) \qquad (1)$$

This volume corresponds to the area between T_1 and $T_1 + \Delta T$ in Figure 24. During periods of low flow (i.e. the instantaneous flow is less than the average flow) the stored volume is discharged to maintain average flow conditions.

The evaluation of Equation 1 requires prior knowledge of the peaking period. An examination of flow chart records revealed that the peaking period is 0.5 ± 0.125 day (12 ± 3 hours) in most cases. Using the average value of 0.5 day, the integral term in Equation 1 is related to, by definition, the 0.5 day (12 hour) average peak flow ($\bar{Q}_{12\ hr}$) in the following manner:

$$\int_{T_1}^{T_1 + 0.5} Q\,dt = 0.5\,\bar{Q}_{12\ hr} \qquad (2)$$

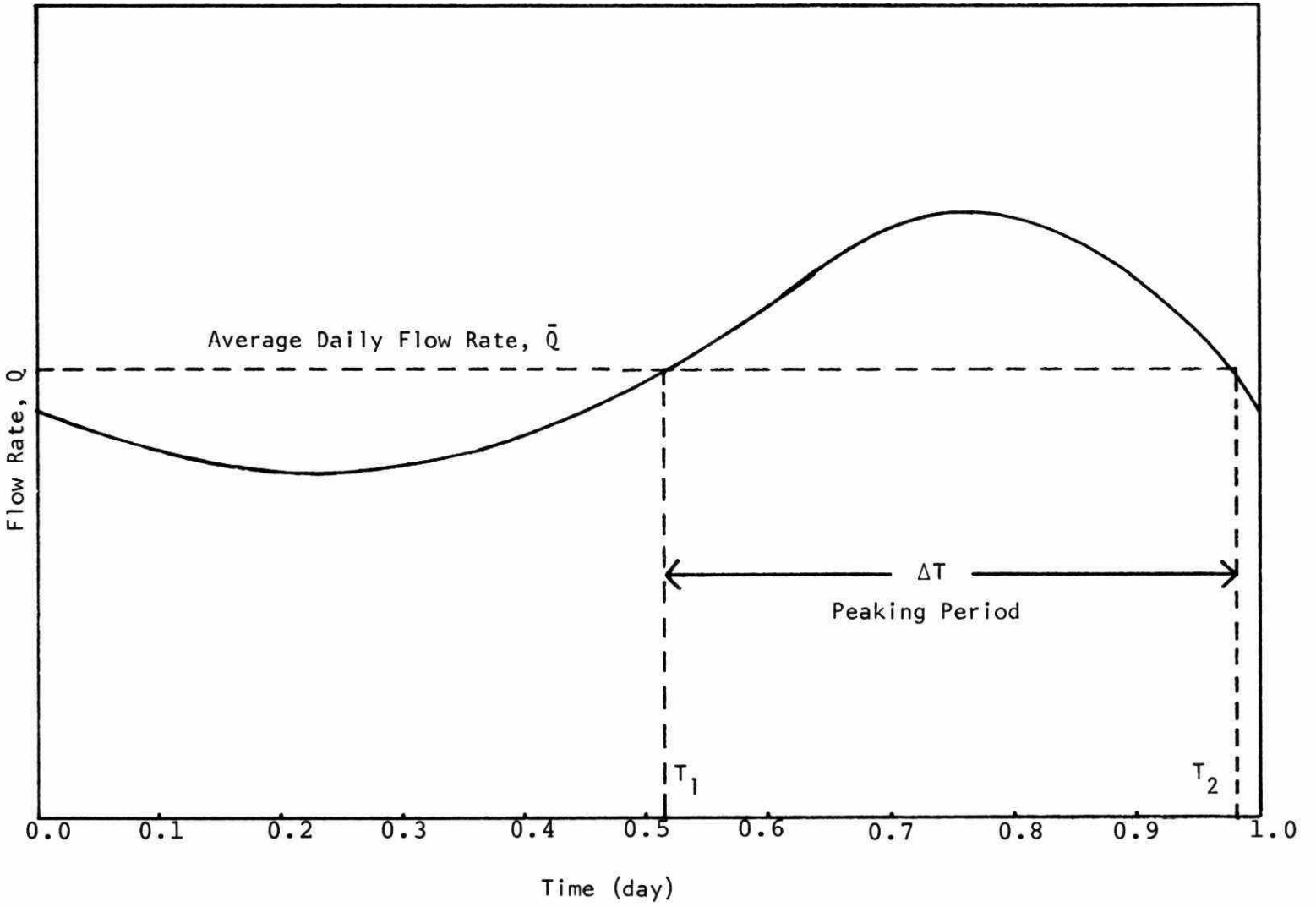


FIGURE 24. IDEALIZED INSTANTANEOUS FLOW VARIATION

where $\bar{Q}_{12\ hr}$ is the average flow for a 12-hour peaking interval.

In terms of the 12-hour diurnal peaking factor the equation for the equalization volume can be rearranged to give:

$$V = 0.5\ \bar{Q}\ (PFd - 1) \tag{3}$$

since

$$PFd\ (12\ hours) = \frac{\bar{Q}_{12\ hr}}{\bar{Q}}$$

The average daily flow rate ($\bar{Q}$) for any selected day can be related to the mean annual daily flow Q_{MADF} by the following equality:

$$\bar{Q} = (PF)\ \bar{Q}_{MADF} \tag{4}$$

In the above equation the proportionality constant is the daily peaking factor, PF.

The combination of Equations 3 and 4 yields an expression for the desired equalization volume as a function of the mean annual daily flow, the daily peaking factor, and the diurnal peaking factor as the following:

$$V = 0.5\ (PF)\ \ (PF_d - 1)\ \bar{Q}_{MADF} \tag{5}$$

Equation 5 was used for the estimation of storage volume requirements. Utilizing the daily and diurnal peaking factors established in Section 3 (see Tables 3 and 8), the equalization volumes for a 5 mgd sewage treatment plant were calculated. The results are summarized in Table 10.

According to Table 10, full equalization requires storage volume of 11% of the mean annual daily flow, Q_{MADF}. This represents an average value, since the parameters were derived from composite data of several plants. Some variations due to specific local sewage management are expected to exist. Nevertheless, it compares favourably with previous estimates of 12.5% [2] and 12% [3] derived by other methods.

The methodology allowed the direct estimation of basin volumes required for partial equalization. In this case the parameter values were based on the flow pattern at a particular exceedence level. For example, if 90% of the daily flows throughout the year were to be

TABLE 10. EQUALIZATION BASIN VOLUMES FOR 5 AND 50 mgd SEWAGE TREATMENT PLANTS

% Equalization	PF	PFD	5 mgd Plant (MG)	5 mgd Plant (cu m)	50 mgd Plant (MG)	50 mgd Plant (cu m)
100%	1.7	1.13	0.552	2,090	5.52	20,900
90%	1.3	1.14	0.455	1,700	4.55	17,000
75%	1.12	1.16	0.438	1,650	4.38	16,500

equalized, then the equalization basin would be designed on the basis of data for 10% exceedence flow. The interpretation of 90% equalization is, of course, that full equalization can be achieved 90% of the time.

During the development of the methodology for selecting equalization basin volumes the diurnal flow peaking period was chosen as 0.5 day in all cases. It is realized that the true peaking period can vary considerably from this value. However, small deviation of an individual peaking period from 12 hr, say ± 3 hr, has little effect (less than 10%) on the estimated equalization basin volume. This is due to the decrease of the diurnal peaking factor with increasing peaking periods. The combination of the peaking period with the diurnal peaking factor (see Equation 5) reduces the error due to the selection of a uniform peaking period of 0.5 day.

Previous methodology for designing equalization basins required the examination of daily and diurnal peaking factors over extended time periods. However, the conclusions concerning flow fluctuations in seven sewage treatment plants suggest less extensive analysis of raw data. The daily and diurnal peaking factors were shown to be independent of plant size. Consequently, the equalization volume can be established by determining the mean annual daily flow and by analyzing a few daily flow charts.

The equalization volumes given in Figure 24 denote minimal volumes for typical sewage treatment plants. To account for plant to plant and annual variability, the incorporation of a safety factor is desirable. Using a recommended safety factor of 1.25 [3], the empirical design volumes become 14%, 13% and 11% of the mean annual daily flow

at full, 90% and 75% equalization, respectively. Previously the agitation and the aeration of proposed equalization basins has been recommended to prevent solids settling and odour formation [2, 3, 4]. In this report diffused aeration is used for both mixing and air input based on a demand of 2 cfm (cubic feet per minute) per ft length of the equalization tank.

4.3 Primary Sedimentation

The costing of primary sedimentation tanks is normally based on the clarifier area. Other variables such as the depth and the geometric configuration of the tank are selected to conform with established design practice. The clarifier area, in turn, can be related to the sewage flow and a chosen overflow rate. Usually the primary sedimentation facility is designed to exclude bypassing. Consequently, the clarifier area is established for a peak hydraulic capacity. The equation relating the clarifier area to the maximum flow rate and the overflow rate is the following:

$$A = \frac{\bar{Q}_{MAX}}{OR} \tag{6}$$

where: A = clarifier area

$\bar{Q}_{MAX}$ = maximum flow rate

OR = overflow rate.

In general practice, the calculation is initiated by establishing criteria for the overflow rate. Based on past experience, the overflow rate should be in the range of 1000 - 1200 gpd/ft^2 (41 - 49 cu m/day/sq m) under maximum and 600 - 800 gpd/ft^2 (24.5 - 32.6 cu m/day/sq m) under mean daily flow conditions. In this report the overflow rate is selected to be 1200 gpd/ft^2 corresponding to the 12-hour maximum hydraulic flow (i.e. 12-hour peak flow on the maximum day).

The next step is the estimation of the 12-hour maximum flow. It has been shown in Section 3.4 that the hydraulic flow can be expressed in terms of the mean annual daily flow, the daily, and the diurnal peaking factors. In the absence of equalization the sewage flow is given by the following equation:

$$\bar{Q}_{MAX} = PF \times PFd \times \bar{Q}_{MADF} \tag{7}$$

The incorporation of an equalization basin results in the attenuation of Q_{MAX}. Utilizing the storage capacity provided by full or partial equalization, the equation for the 12-hour maximum flow becomes:

$$\bar{Q}_{MAX} = PF \times PFd \times \bar{Q}_{MADF} - \frac{Ve}{0.5 \text{ day}} \quad (8)$$

where: $\bar{Q}_{MAX}$ = 12-hour maximum flow (MGD)

PF = daily peaking factor

PFd = diurnal peaking factor for 12-hour peaking

$\bar{Q}_{MADF}$ = mean annual daily flow

Ve = equalization basin volume.

When full equalization is practised, the equalization basin volume is given by $0.5 \times PF \times (PFd - 1) \times \bar{Q}_{MADF}$ (see Equation 5 Section 4.2). By substituting into Equation 8, the 12-hour maximum flow during full equalization practice becomes:

$$\bar{Q}_{MAX} = PF \times \bar{Q}_{MADF} \quad (9)$$

As expected, the effect of the diurnal peaking factor is negated. The 12-hour maximum flow is dependent on the daily peaking factor only.

When partial equalization is used, the 12-hour maximum flow ($\bar{Q}_{MAX}$) can be readily obtained by substituting the established storage volume into Equation 8. In this case the peak design flow for the primary sedimentation tank lies between the two criteria given above.

Using the established parameters for the OR and $\bar{Q}_{MAX}$ the clarifier area can be readily calculated for the 5 mgd and the 50 mgd plant. The results are summarized in Table 11.

4.4 Aeration Tanks

Present practice for variable flow plants is to design the aeration stage corresponding to annual average flow conditions. The same criterion will be applied for plants with varying degrees of equalization. The most important criterion for the design is the organic loading defined by the following equation:

$$\frac{F}{M} = \frac{BOD_{IN} \text{ (lb)}}{MLVSS \text{ (lb)}} \text{ DT (day)} \quad (10)$$

TABLE 11. DESIGN CRITERIA FOR PRIMARY SEDIMENTATION TANKS

% Equalization	5 mgd Plant		50 mgd Plant		Overflow Rate 12-hr Max. Flow gpd/ft^2 (cu m/day/ sq/m)	Overflow Rate Mean Annual Daily Flow gpd/ft^2 (cu m/day/ sq/m)
	12-hr Max. Flow, mgd (cu m/day)	Clarifier Area, ft^2 (sq m)	12-hr Max. Flow, mgd (cu m/day)	Clarifier Area, ft^2 (sq m)		
0	9,605 (36,400)	8,000 (740)	96.05 (364,000)	80,000 (7,400)	1,200 (49)	625 (25.5)
75	8,729 (33,000)	7,274 (680)	87.29 (330,000)	72,740 (6,800)	1,200 (49)	687 (28.0)
90	8,695 (32,900)	7,245 (670)	86.95 (329,000)	72,450 (6,700)	1,200 (49)	690 (28.2)
100	8,500 (32,200)	7,084 (660)	85.00 (322,000)	70,840 (6,600)	1,200 (49)	706 (28.8)

where: BOD_{IN} = BOD in inflow to the aeration tank

MLVSS = mass of MLVSS in the tank

DT = hydraulic detention time.

Past experience indicates that a BOD loading of 35 lb per 100 lb MLVSS per day (0.35 kg/kg/day) appears to be the optimum in municipal waste treatment systems. Therefore, an F/M ratio of 0.35 day^{-1} was used for design calculations. The volume of the aeration tanks is given by:

$$V = \frac{BOD_{IN\ (CONC.)}}{MLVSS \times F/M} \qquad \bar{Q}_{MADF} \tag{11}$$

In the above equation BOD_{IN} represents the actual BOD concentration in the tank influent. To solve the equation, information concerning the removal of BOD in the primary stage is required. Raw sewage is assumed to have a BOD of 200 mg/l. Under variable flow conditions 20% BOD removed in the primary stage can be used as a conservative estimate. It has been shown [5] that under equalized flow conditions the BOD removal in the primary stage improves significantly yielding an overall BOD reduction of 35%. Thus, the BOD concentrations under variable and fully equalized flow conditions are 160 mg/l and 130 mg/l, respectively. The effect of partial equalization on BOD removal is estimated by linear interpolation between the above values. The MLVSS concentration is assumed to be 2,000 mg/l for both variable flow and equalized plants. This concentration corresponds to the reported value for optimal efficiency. The calculated volumes for the aeration tanks are summarized in Table 12.

TABLE 12. AERATION TANK VOLUMES FOR 5 mgd AND 50 mgd PLANTS

% Equalization	5 mgd Plant (MG)	(cu m)	50 mgd Plant (MG)	(cu m)
0	1.14	4,300	11.4	43,000
75	1.00	3,800	10.0	38,000
90	0.96	3,600	9.6	36,000
100	0.93	3,500	9.3	35,000

4.5 Air Blowers (Aeration)

The oxygen requirement may be established from the following equation:

$$Sa = 6.95\ Q_{MAX}\ BOD_R \text{ (cu ft/min)} \quad (12)$$

where: Q_{MAX} = 12 hr maximum average flow

BOD_R = BOD oxidized in the aeration tank.

For estimating the size of the aeration equipment the 12-hour maximum flows given in Table 11 were used. The BOD in the effluent from the activated sludge unit was assumed to be 15 mg/l in all cases. The aeration requirement is summarized in Table 13. The cost of air supply equipment is based on the aeration requirement given in the table.

TABLE 13. AERATION REQUIREMENT FOR 5 mgd AND 50 mgd PLANTS

		5 mgd Plant		50 mgd Plant	
% Equalization	BOD_R mg/l	Q_{MAX} mgd (cu m/day)	Aeration cu ft/min (cu m/min)	Q_{MAX} mgd (cu m/day)	Aeration cu ft/min (cu m/min)
0	145	9,605 (36.358)	9,800 (278)	96.05 (363,587)	98,000 (2780)
75	125	8,729 (33.042)	7,600 (215)	87.29 (330,427)	76,000 (2150)
90	120	8,695 (32.914)	7,250 (205)	86.95 (329,140)	72,500 (2050)
100	115	8,500 (32.175)	6,800 (192)	8,500 (321,759)	68,000 (1920)

4.6 Return Sludge Pumping

The return sludge pumping capacity required is based on an approximate mass balance carried out on the aeration unit:

$$\bar{Q}_R \text{ (mgd)} = \bar{Q}_{MAX} \frac{\text{(mgd) MLVSS (mg/l)}}{C_S - \text{MLVSS (mg/l)}}$$

where: $\bar{Q}_R$ = maximum return sludge pumping capacity (mgd)

C_S = return sludge concentration in mg/l

Q_{MAX} = 12-hour maximum flow.

In this report a C_S of 6,000 mg/l is assumed at maximum design flow conditions. A MLVSS of 2,000 mg/l will be used in all cases. The calculated pumping capacity is given in Table 14.

TABLE 14. RETURN SLUDGE PUMPING CAPACITY

	5 mgd Plant		50 mgd Plant	
% Equalization	Q_{MAX}, mgd (cu m/day)	Pumping Cap., mgd (cu m/day)	Q_{MAX}, mgd (cu m/day)	Pumping Cap., mgd (cu m/day)
0	9,605 (36,358)	4.80 (18,200)	96.05 (363,587)	48.0 (182,000)
75	8,729 (33,042)	4.40 (16,700)	87.29 (330,427)	44.0 (167,000)
90	8,695 (32,914)	4.30 (16,300)	86.95 (329,140)	43.0 (163,000)
100	8,500 (32,175)	4.20 (15,900)	85.00 (321,759)	42.0 (159,000)

4.7 Secondary Sedimentation Tanks

The performance of the secondary sedimentation stage is critical in a secondary treatment plant because it directly influences the quality of the plant effluent.

Traditionally the sizing of the secondary sedimentation tanks is based on a maximum overflow rate of 800 gpd/ft^2. The methodology for calculating the desired tank area was identical to the estimation of primary sedimentation tank area. The results are summarized in Table 15.

TABLE 15. SECONDARY SEDIMENTATION TANK AREA

% Equalization	5 mgd Plant Q_{MAX} mgd (cu m/day)	5 mgd Plant Area sq ft (sq m)	50 mgd Plant Q_{MAX} mgd (cu m/day)	50 mgd Plant Area sq ft (sq m)
0	9,605 (36.358)	12,000 (1114.836)	96.05 (363.587)	120,000 (11148.36)
75	8,729 (33.042)	10,900 (1012.642)	87.29 (330.427)	109,000 (10126.42)
90	8,695 (32.914)	10,870 (1009.855)	86.95 (329,140)	108,700 (10098.55)
100	8,500 (32.1759)	10,600 (984.71)	85.00 (321.759)	106,000 (9847.71)

4.8 Chlorination

The amount of chlorine required is taken as 4.5 mg/l (40 lb/MG) based on the $\bar{Q}_{ADF}$ for the plant. Therefore, it is the same for all plants of the same size.

The chlorination chambers, however, are based on a minimum hydraulic detention time of 15 minutes at peak flow to the chlorinator. Accordingly any decrease in the peak flow will decrease the tank size and associated construction costs. This is the case for fully and partly equalized flow to the plant. The size of chlorinators is summarized in Table 16.

TABLE 16. CHLORINATION BASIN SIZE

% Equalization	5 mgd Plant Q_{MAX} mgd (cu m/day)	5 mgd Plant Volume MG (cu m)	50 mgd Plant Q_{MAX} mgd (cu m/day)	50 mgd Plant Volume MG (cu m)
0	9.605 (36,358)	0.100 (378)	96.05 (363,587)	1.00 (3,780)
75	8.729 (33,092)	0.091 (344)	87.29 (330,427)	0.91 (3,440)
90	8.695 (32,914)	0.091 (344)	86.95 (329,140)	0.91 (3,440)
100	8.500 (32,175)	0.089 (337)	85.00 (321,759)	0.89 (3,370)

4.9 Cost Comparisons

1. Variable Flow - No bypassing
2. Fully Equalized Flow - No bypassing
3. Partly Equalized Flow - No bypassing

For ease of comparison the relative construction costs are listed in Tables 17 and 18 for the main units of a traditional sewage treatment plant. This table lists those units only which may be affected by the change of flow conditions for two plant sizes (5 mgd and 50 mgd as $\bar{Q}_{ADF}$).

The cost estimates are based on the US-EPA's publication, "Estimating Costs and Manpower Requirements for Conventional Wastewater Treatment Facilities" (1971). These cost figures were updated to July, 1974 figures based on a CCI increase of 50% for Toronto, Ontario. It should be kept in mind that the costs given are based, in part, on theoretical estimated values and extrapolations from the above report. The estimate does not include land area costs. This may result in distorted total costs. However, the relative costs should not be greatly affected.

From the tabulated data it is apparent that the introduction of equalization results in small reduction of construction costs. A 5% cost benefit may be achieved by full equalization in the 5 mgd and the 50 mgd plants. Partial equalization does not appear to be attractive economically. The saving in size reduction is negated by the incremental cost of the equalization basin.

4.10 Relative Operating Cost

The operating costs for variable flow relative to equalized flow sewage treatment plants can not be established accurately. However, the labour cost is not expected to differ greatly.

In smaller plants which do not have automated operations at the present, the automation of the equalization basin operation may add to operating complexity. However, the operation of subsequent treatment stages may be simplified greatly.

The capacity of equipment and each unit process operation will be more fully utilized when operating at equalized flow. A decrease

TABLE 17. ESTIMATED CONSTRUCTION COST 5 mgd PLANT

Cost Component	0% Equalization		75% Equalization		90% Equalization		100% Equalization	
	Size	Cost $ x 1000	Size	Cost $ x 1000	Size	Cost $ x 1000	Size	Cost $ x 1000
Equalization Basin, cu ft (cu m)	-	-	70,300 (1,650)	132	73,000 (1,700)	135	88,600 (2,090)	156
Primary Sed. Tank, sq ft (sq m)	8,000 (740)	180	7,300 (680)	173	7,200 (670)	173	7,000 (660)	170
Aeration Basin, cu ft (cu m)	183,000 (4,300)	288	161,000 (3,800)	258	154,000 (3,600)	245	149,000 (3,500)	240
Aeration Diffused Air, cfm (cu m/min)	9,800 (278)	465	7,600 (215)	375	7,300 (205)	360	6,800 (192)	345
Return Sludge Pumping, mgd (cu m/day)	4.8 (18,200)	108	4.4 (16,700)	101	4.3 (16,300)	98	4.2 (15,900)	96
Secondary Sed. Tank, sq ft (sq m)	12,000 (1,100)	225	10,900 (1,010)	210	10,900 (1,000)	210	10,600 (980)	203
Chlorination Basin, cu ft (cu m)	16,000 (378)	54	14,600 (344)	51	14,600 (344)	51	14,300 (337)	50
TOTAL		1,320		1,300		1,272		1,260
Relative Cost		100.0		98.5		96.4		95.5

TABLE 18. ESTIMATED CONSTRUCTION COST 50 mgd PLANT

Cost Component	0% Equalization		75% Equalization		90% Equalization		100% Equalization	
	Size	Cost $ x 1000	Size	Cost $ x 1000	Size	Cost $ x 1000	Size	Cost $ x 1000
Equalization Basin, cu ft (cu m)	-	-	703,000 (16,500)	930	730,000 (17,000)	960	886,000 (20,900)	1,170
Primary Sed. Tank, sq ft (sq m)	80,000 (7,400)	1,155	73,000 (6,800)	1,065	73,000 (6,700)	1,065	70,000 (6,600)	1,005
Aeration Basin, cu ft (cu m)	1,830,000 (43,000)	2,200	1,610,000 (38,000)	2,100	1,540,000 (36,000)	2,030	1,490,000 (35,000)	950
Aeration Diffused Air, cfm (cu m/min)	98,000 (2,780)	2,700	76,000 (2,150)	2,180	73,000 (2,050)	2,030	68,000 (1,920)	900
Return Sludge Pumping, mgd (cu m/day)	48 (182,000)	600	44 (167,000)	540	43 (163,000)	530	42 (159,000)	525
Secondary Sed. Tank, sq ft (sq m)	120,000 (11,000)	1,725	109,000 (10,100)	1,470	109,000 (10,000)	1,470	106,000 (9,800)	1,400
Chlorination Basin, cu ft (cu m)	160,000 (3,780)	270	146,000 (3,440)	250	146,000 (3,440)	250	143,000 (3,370)	240
TOTAL		8,650		8,535		8,335		8,190
Relative Cost		100		98.7		96.4		94.7

in peak power requirement of pumps and air blowers relative to average operating conditions may decrease the energy cost on a per flow basis due to surcharges.

With a greater utilization of blowers and pumps it is likely that each unit will require more frequent servicing. However, the maintenance per volume treated may be unchanged.

If control operations in the plant are presently automated it is expected that control valves, pumps and air blowers will need fewer control instructions per day due to the more uniform flow conditions. Thus, less energy will be used for control operations and less wear would be expected for the control valves, etc.

It is expected that the performance of sedimentation and aeration stages will improve for equalized flow relative to variable flow plants, resulting in a better plant effluent, or in a smaller plant to achieve equivalent effluent quality. The improvement in the biological treatment stage has not been accounted for in the estimates of the previous section.

5. CONCLUSIONS

To establish meaningful design criteria for equalization, extensive analysis of daily and diurnal variations of flow in Ontario sewage treatment plants was carried out. The flow variations were accounted for by daily and diurnal peaking factors. A significant conclusion of the analysis was that the peaking factors are essentially independent of plant size. Peaking factors for individual plants, however, varied considerably. These variations were believed to reflect differences in plant collection systems, the most important factor being the combination of sanitary sewage with storm runoff.

Flow equalization is a means of eliminating flow bypassing at the sewage treatment plant. The most important criteria for flow equalization appeared to be the maximum daily peaking factor and the diurnal peaking factor. As expected, the diurnal peaking factors decreased with increasing peaking periods. The decrease was particularly pronounced during high daily average flow conditions (wet weather).

On the basis of the above mentioned findings a new and simplified method was suggested for sizing and operation of equalization basins. The volume of the basin was estimated to be 11% to 14% of the mean annual daily flow. This volume size ensured continuous operation without overflow.

Maximum benefit due to equalization can be predicted for the performance of primary clarifiers. The equalization would result in significant reduction of suspended solids variations relative to raw sewage. Due to high level of microbial activity, the equalization basin should be aerated and agitated for the prevention of anaerobiosis and accompanying odours.

A construction cost comparison for two plant sizes (5 and 50 mgd mean annual daily flow) indicated that fully equalized flow can result in a small reduction over variable flow design. Partial equalization does not appear to be a viable alternative. It is felt that the cost benefit is a conservative estimate. Potential cost benefits may be even higher when increased performance of the various units and the elimination of surcharge energy costs at peak loadings are included.

The accuracy of the cost and design evaluations in this report can only be improved if more full scale data on the operation and performance

of equalized and variable flow conditions becomes available. Obtaining such data would be costly unless an existing multistream sewage treatment plant may be manipulated without deterioration of overall plant performance. It is recommended, therefore, that full scale studies be conducted in existing multistream wastewater treatment plants with parallel trains of treatment units. The benefits on each treatment stage can be assessed by equalizing the flow in one of the streams.

REFERENCES

1. James F. MacLaren Limited, "To Establish Viable Methods of Maintaining Waste Treatment Facility Efficiencies with Reference to Flow Variations", Volume I, Canada-Ontario Agreement Research Report No. 11, Ottawa, 1974.

2. LaGrega, M.D. and Keenan, J.D., "Effects of Equalizing Wastewater Flows", JWPCP 46(1):123-132, January, 1974.

3. Smith, R., Eilers, R.E. and Hall, E.D., "Design and Simulation of Equalization Basins", National Environmental Research Centre US EPA, National Environmental Research Centre Report #EPA-670/2-73-046, 1973.

4. McDuff, D.P. and Ward, D.B., "Plant Expansion for Advanced Waste Treatment", Public Works 105(3):56-59, March, 1974.

5. LaGrega, M.D., "A Study of the Effects of Equalization of Wastewater Flows", Ph.D. Dissertation, Syracuse University, 1972.

6. Edmonds, Dr. C.J. et al, "To Establish Viable Methods of Maintaining Waste Treatment Facility Efficiencies With Reference to Waste Variations", Paper presented at 4th Joint Chem. Eng. Conference (AIChE + CSChE), Vancouver, B.C. September, 1973.

APPENDIX I

NOMENCLATURE

APPENDIX I

NOMENCLATURE

A	Surface area of clarifier
BOD_{IN}	Biochemical oxygen demand in the influent stream
BOD_{OUT}	Biochemocal oxygen demand in the effluent stream
BOD_R	Biochemical oxygen demand removed in the unit
C_S	Concentration of solids in waste activated sludge
DT	Detention time
F/M	Organic loading
PF	Daily peaking factor
PFd	Diurnal peaking factor
Q	Instantaneous flow rate
$\bar{Q}$	Average flow rate
$\bar{Q}_{MADF}$	Mean annual daily flow rate
$\bar{Q}_{MAX}$	12-hour maximum flow rate
Q_R	Sludge return flow rate
Sa	Aeration rate
T	Time period
V	Volume
V_e	Equalization basin volume

APPENDIX II

ANALYSIS OF DAILY FLOW VARIATIONS

APPENDIX II

ANALYSIS OF DAILY FLOW VARIATIONS

The statistical analysis of the daily flow rates was accomplished with the aid of an in-house computer program. Flow data for one year time periods were stored on a disk file. The data were scanned to establish the probability of exceedence. The print-out included exceedence levels from 0% to 100%. Minimum exceedence level corresponded to maximum daily flow.

Graphical representations of the data analysis are shown in Figures 25 to 30. The curves represent percent exceedence as a function of the daily flow for two sewage treatment systems. On these figures the exceedence flows used for calculating the daily peaking factors are illustrated. Similar plots have been prepared for all sewage treatment plants considered in this study.

FIGURE 25. FREQUENCY OF EXCEEDENCE OF DAILY MEAN FLOWS

MUNICIPALITY: D

YEAR: 1971

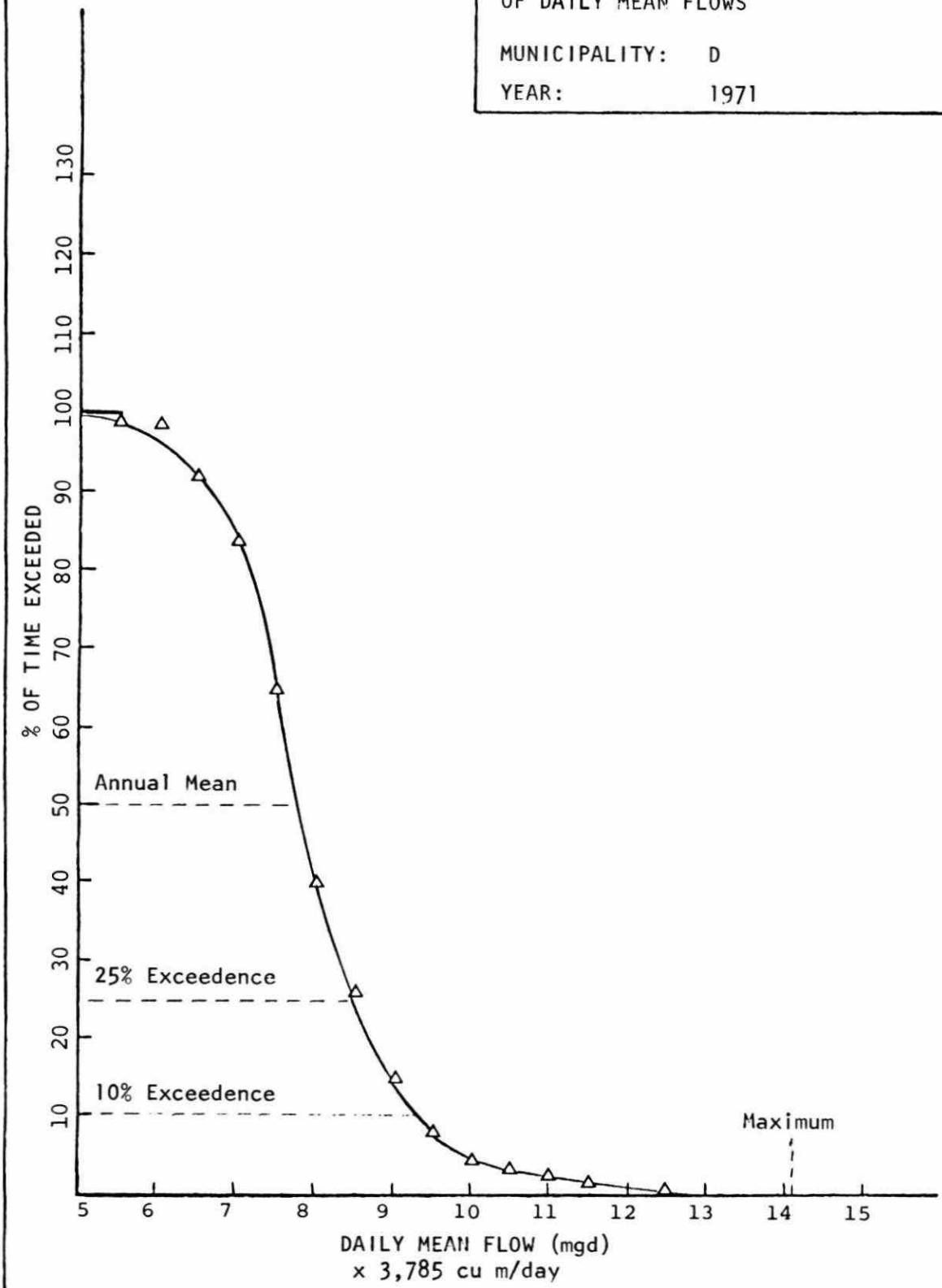


FIGURE 26. FREQUENCY OF EXCEEDENCE OF DAILY MEAN FLOWS

MUNICIPALITY: D

YEAR: 1972

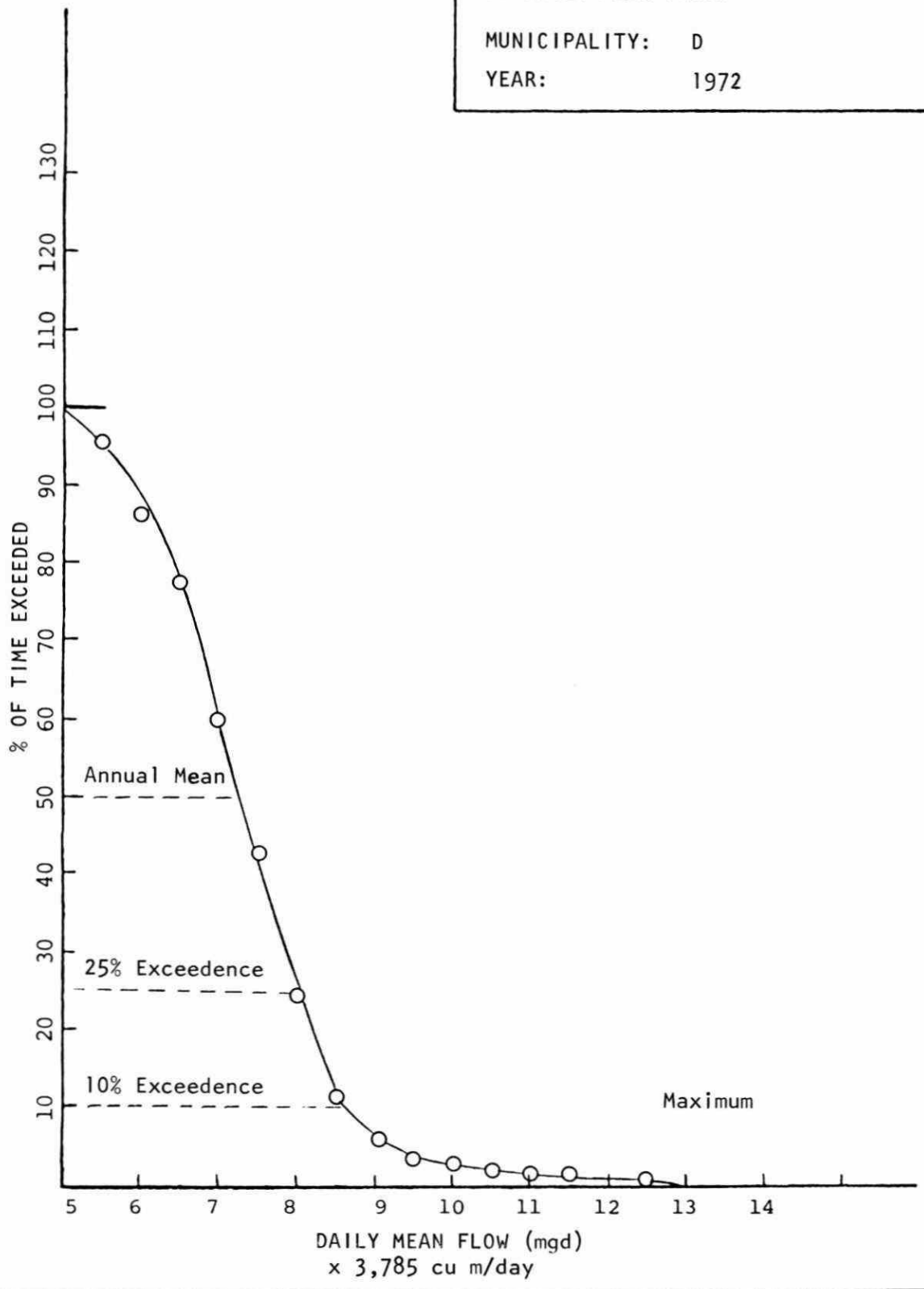


FIGURE 27. FREQUENCY OF EXCEEDENCE OF DAILY MEAN FLOWS

MUNICIPALITY: D

YEAR: 1973

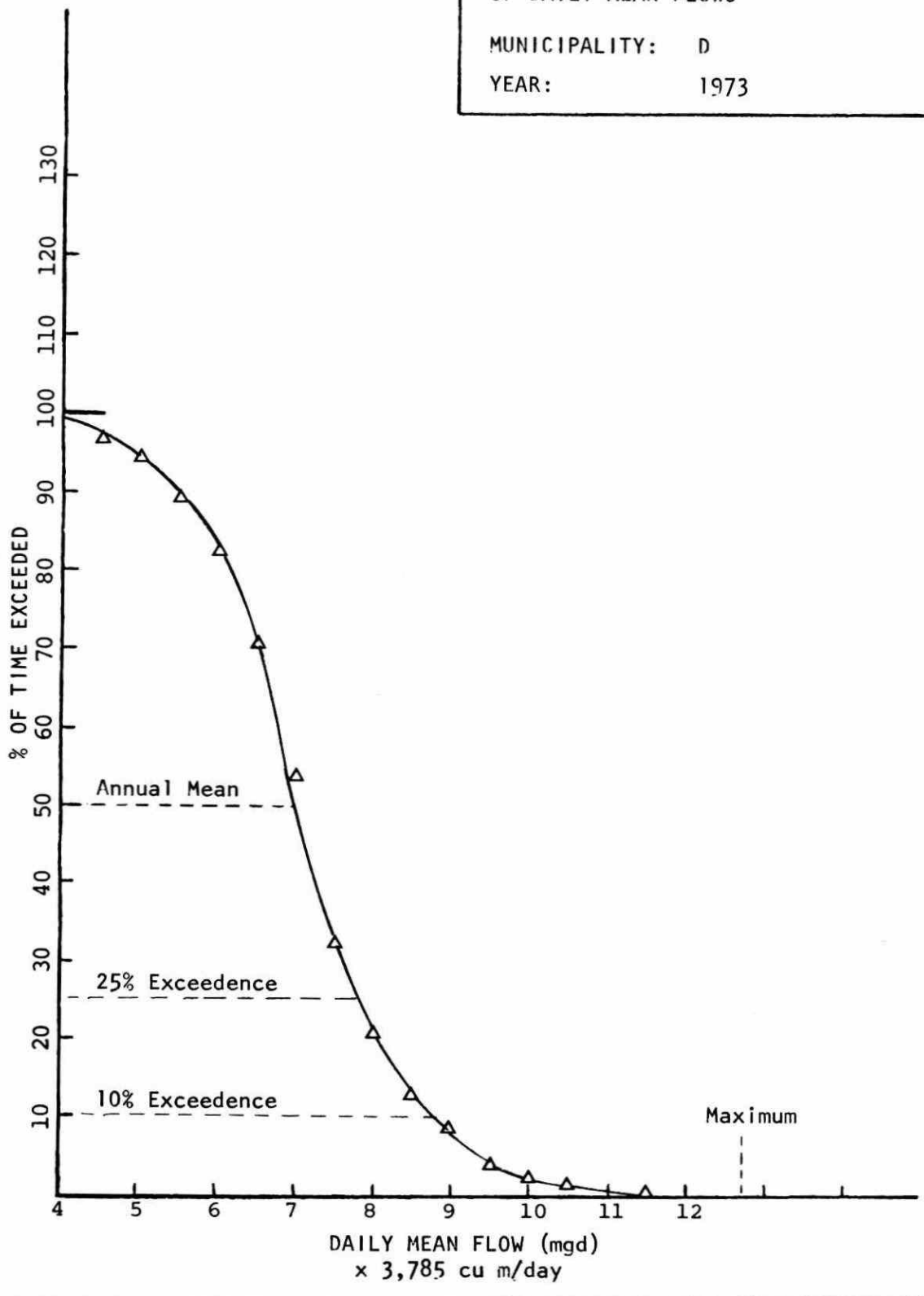


FIGURE 28. FREQUENCY OF EXCEEDENCE OF DAILY MEAN FLOWS

MUNICIPALITY: E

YEAR: 1970

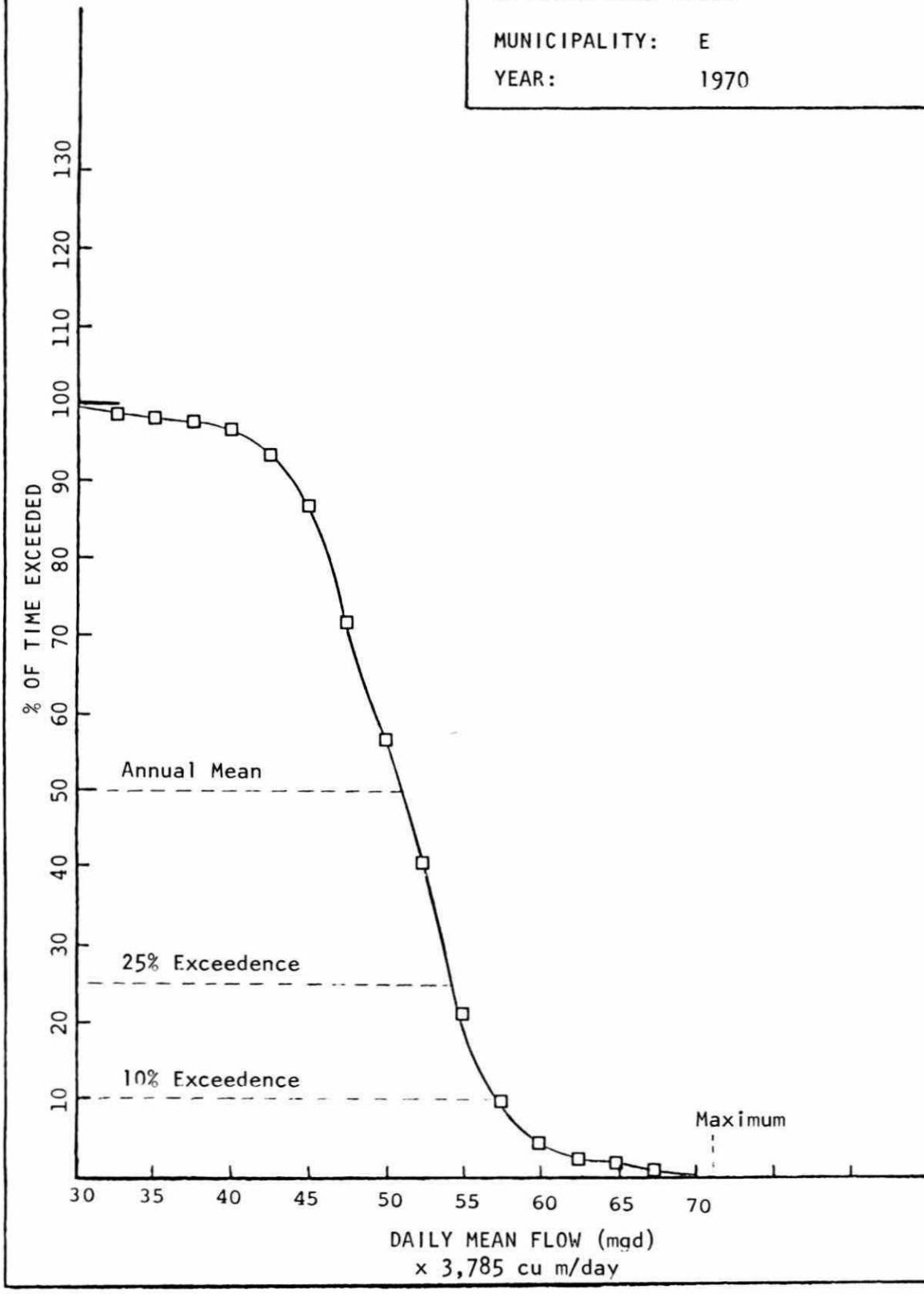


FIGURE 29. FREQUENCY OF EXCEEDENCE OF DAILY MEAN FLOWS
MUNICIPALITY: E
YEAR: 1971

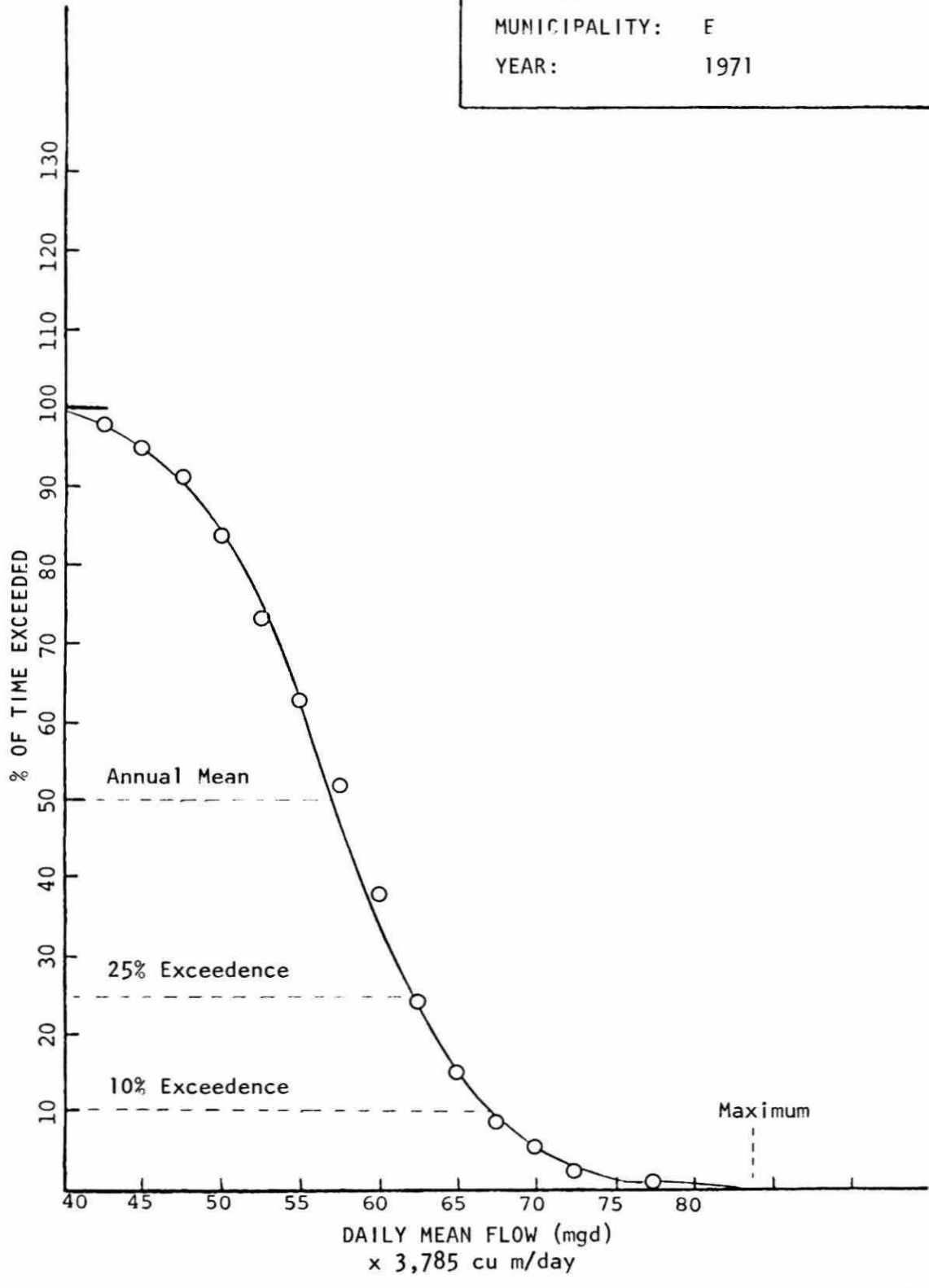
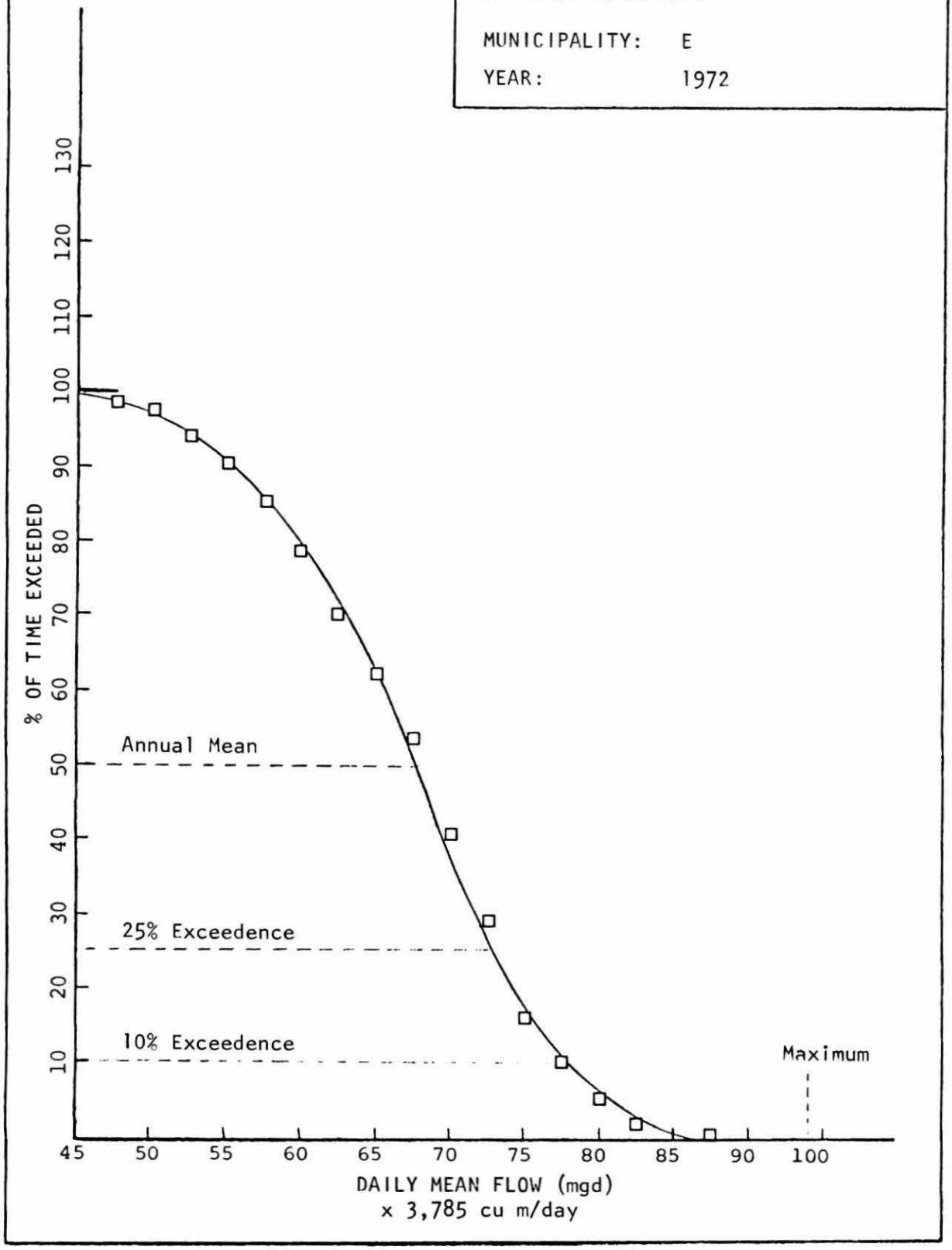


FIGURE 30. FREQUENCY OF EXCEEDENCE OF DAILY MEAN FLOWS

MUNICIPALITY: E

YEAR: 1972



TD
746
.E88
1977

To establish viable methods of maintaining waste treatment facility efficiencies with reference to flow variations :

78482